IRC:73-2023

GEOMETRIC DESIGN STANDARDS FOR NON-URBAN ROADS

(First Revision)

INDIAN ROADS CONGRESS
2023

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ABBREVIATIONS & SYMBOLS

All the abbreviations and symbols are explained in the guidelines wherever they appeared first in the document. Some of the important abbreviations and symbols are listed below:

AASHTO - American Association of State Highway and Transportation Officials

ADT - Average Daily Traffic

ADV - Animal Drawn Vehicle

C - Allowable Rate of Change of Centrifugal Acceleration

CUP - Cattle Underpass

d_{SSD} - Minimum Stopping Sight Distance Required

e - Super-Elevation

E_s - Apex Distance

f - Coefficient of Longitudinal Friction between Vehicle Tyres and Road Pavement

g - Acceleration due to Gravity

GC - Grade Compensation

H - Height of Driver's Eye

h - Object Height

H.I.P - Horizontal Intersection Point

HFL - High Flood Level

HL - Horizontal Alignment

HSD - Headlight Sight Distance

INDO-HCM - Indian Highway Capacity Manual

IRC - Indian Roads Congress

IRC:SP - Indian Roads Congress: Special Publication

ISD - Intermediate Sight Distance

L - Length of Wheel Base of Longest Vehicle

L_C - Length of the Horizontal Curve

LOS - Level of Service

L_s - Length of Transition Curve

LVUP - Light Vehicular Underpass

m - Minimum Setback Distance to Sight Obstruction in Meters at the Middle of the Curve

M - Minimum Ordinate

MDR - Major District Roads

N	-	The Deviation Angle
NH	-	National Highways
ODR	-	Other District Roads
OSD	-	Overtaking Sight Distance
P	-	Centrifugal Force
PCU	-	Passenger Car Unit
PIEV	-	Perception Intellection Emotion Volition
PS	-	Paved Shoulders
PUP	-	Pedestrian Underpass
R	-	Radius at the Centre Line of the Road
RC	-	Radius of Circular Curve
ROB	-	Road Over Bridge
S	-	Sight Distance
s	-	Shift
SH	-	State Highways
SSD	-	Stopping Sight Distance
SVUP	-	Small Vehicular Underpass
t	-	Perception and Reaction Time
T.P. or P.T.	-	Tangent Point
T_s	-	Tangent Distance
V	-	Design Speed
VIP	-	Vertical Point of Intersection
VL	-	Vertical Alignment
VOP	-	Vehicular Overpass
VR	-	Village Roads
VUP	-	Vehicular Underpass
W	-	Weight of the Vehicle
W_e	-	Extra Width on Curves
X_0	-	Highest or Lowest Point on Summit or Valley Curves
Δ	-	Total Deviation Angle
Δ_c	-	Deviation and Central Angle of Circular
θ_s	-	Deviation Angle of Transition Curve
θ	-	Half Central Angle

GEOMETRIC DESIGN STANDARDS FOR NON-URBAN ROADS

1. INTRODUCTION

1.1 Background

IRC:73 was first published during the year 1980. Over the years, due to drastic changes in the geometric design standards as well as in the design concepts of non-urban highways (rural highways), need was felt to revise these standards. The ambitious task of revising IRC:73-1980 included merging the following guidelines in unison.

- IRC:66-1976 "Recommended Practice for Sight Distance on Rural Highways"
- IRC:SP:23-1983 "Vertical Curves for Highways"
- IRC:32-1969 "Standard Vertical and Horizontal Clearances of Overhead Electric Power and Telecommunication Lines as related to Roads"

In addition, IRC:54-1974 "Lateral and Vertical Clearances at Underpasses for Vehicular Traffic" have been included partly (applicable to Rural Highways). The relevant aspects of Geometric Design from IRC:38-1988 "Guidelines for Design of Horizontal Curves for Highways and Design Tables" have been also duly covered and due references have been made to this guideline.

After a thorough discussion, it was decided that the guidelines should be prepared on similar lines of AASHTO Green Book; a zero version is being aimed now and focussed on non-urban highways, while later on urban roads can be combined to make an Indian Green Book. The Transport Planning and Traffic Engineering Committee (H-1) of IRC took up this task of revision during the tenure 2018-20 and a subgroup comprising Dr. Bhargab Maitra (Subgroup Chairman), Dr. Rajat Rastogi, Dr. P.P. Dey, Shri S.S. Khandelwal and Dr. Sudeshna Mitra was constituted. The work of modifying the draft based on enormous comments and suggestions was carried out by Shri Kaustubh Saysardar and Ms. Pidathala Vaishnavi. The draft prepared by the subgroup was deliberated in various meetings of H-1 Committee and was finalized in its meeting held on 29.05.2020. The revised draft document was placed before the Highways Specifications and Standards Committee (HSS) in its meeting held on 12.09.2020 and continued on 19.09.2020 & 03.10.2020. The HSS Committee decided that Convenor, H-1 Committee will modify the document based on written and verbal comments offered during the meeting and submit final document to IRC for placing again to HSS Committee. Thereafter, the draft was modified and deliberated during various meetings of newly constituted H-1 Committee for the tenure 2021-23 and was finalized by the H-1 Committee in its meeting held on 13.09.2022.

The composition of H-1 Committee is given below:

Chandra, Prof. (Dr.) Satish	.....	Convenor
Ram, Prof. (Dr.) Sewa	.....	Co-Convenor
Madhu, Dr. E.	.....	Member-Secretary

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Director General (Road Development) & Special Secretary to Govt. of India	(Nirmal, S.K.), Ministry of Road Transport & Highways
Secretary General, Indian Roads Congress	Nirmal, S.K.

The revised draft document was placed before the HSS Committee in its meeting held on 24.09.2022. The HSS Committee approved this draft revision subject to suitably incorporating written and verbal comments received and also recommended it for placing before the meetings of Executive Committee (EC) and Council. The EC in its meeting held on 07.10.2022 approved the draft revision for placing before the Council. The 223rd Council in its meeting held on 8th October, 2022 at Lucknow (Uttar Pradesh) considered and approved draft revision of IRC:73 "Geometric Design Standards for Non-Urban Roads" for printing.

1.2 General

Geometric design deals with the visible elements of a roadway. Horizontal and vertical alignments together constitute important components in the design of the longitudinal profile of a road. In addition to alignments, sight distance is of profound importance for the safe and efficient operation of vehicles on a roadway. For designing safe roadway alignment, sight distance of adequate length must be ensured in different situations to permit drivers enough time and distance to control their vehicles to avoid unforeseen accidents. Sound geometric design results in economical operation of vehicles and ensures safety.

1.3 Scope

This publication is based primarily on existing standards and recommendations of the Indian Roads Congress, with suitable recent modifications and additions in the light of current engineering practice.

In applying this standard, effort in general should be aimed at standards higher than the minimum indicated and the design of any roadway is not limited in any sense. Where conditions are favourable, good engineering practice will lie in adopting values that are more liberal. The standards prescribed are essentially advisory in nature but may be relaxed somewhat in very difficult situations if considered judicious. As far as possible, uniformity of design standards shall be maintained throughout the length of roads. In case of any change in alignment, it should be implemented in a gradual manner.

The text deals with geometric design standards for rural roads, i.e. non-urban roads located predominantly in open country outside the built-up area. The road alignment may, however, pass through isolated stretches of built-up nature as long as character of the road as a whole does not change. The standard covers geometric design guidelines with respect to Expressways, 2/4/6/8 Lane Highways as well as Major District Roads (MDR), Other District Roads (ODR) and Village Roads (VR). Built up area of the roadway means sections that are situated within municipal limits. Sections of 200 m or more in non-municipal areas where permanent structures are built on one side or both sides of the highway on at least 50% of the total length comprising such section shall be treated as built up area.

Revised IRC:73 guidelines cover the geometric design features for different classification of roads. However, it may not cover the aspects in details. Therefore, the following guidelines and manuals given in **Table 1.1** may be appropriately referred, wherever required.

Table 1.1 List of Guidelines for Additional Details on Geometric Design

Guidelines/Manual	Title
IRC:SP:73	Manual of Specifications & Standards for Two Lanning of Highways with Paved Shoulders
IRC:SP:84	Manual of Specifications & Standards for Four Laning of Highways
IRC:SP:87	Manual of Specifications & Standards for Six Laning of Highways
IRC:SP:99	Manual of Specifications and Standards for Expressways
IRC:52	Guidelines for the Alignment Survey and Geometric Design of Hill Roads
IRC:SP:20	Rural Roads Manual
IRC:38	Guidelines for Design of Horizontal Curves for Highways & Design Tables
IRC:SP:90	Guidelines for Grade Separators & Elevated Structures

The following standards are not applicable in context of urban roads or city streets. Geometric design elements of road intersections are not considered in these guidelines either.

Geometric deficiencies are costly and sometimes impossible to rectify later on due to the subsequent roadside ribbon development. Therefore, it is essential that geometric requirements should be kept in view right in the beginning.

1.4 Overview of the Guideline

Section 2 provides with details of classification of roads based on the function of roads in terms of accessibility and mobility.

Section 3 includes the necessary major and minor factors to be considered before the start of geometric design of non-urban roads.

Section 4 provides details about the recommended standards for various cross sectional elements of rural roads such as right of way, roadway width, shoulders, median, camber, cross-drainage requirements, etc.

Section 5 includes detailed design information on one of the most important geometric design feature, i.e. sight distance. The section provides details on sight distance in various situations on non-urban roads, horizontal curves and vertical curves, intersections, etc.

Section 6 includes standards for radius of circular curves, super-elevation, extra widening, and transition on horizontal curves of rural roads. Concepts of compound curves, reverse curves and hair pin bends have also been discussed in this section.

Section 7 provides details on gradients, deviation angle, types of vertical curves, summit curves, valley curves and design of vertical curves.

Section 8 discusses the elements of horizontal and vertical coordination together in detail. It includes concepts on three dimensional co-ordination and other considerations for coordination of design elements.

Section 9 compiles lateral and vertical clearances for underpasses, overpasses, electric power and telecommunication lines applicable to single lane roads, intermediate lane roads, two lane roads, multilane roads and expressways. The section also covers standards related to overhead electric power and telecommunication lines erected within the road land.

Section 10 deals in brief with control of access on rural roads. The relevant recommendations regarding access to residential properties, grade separations across roads and access to fuel stations are provided in this section.

2. CLASSIFICATION OF NON-URBAN ROADS

2.1 General

Road classification is an essential tool for rational assessment and assignment of responsibilities in the road sector. It is important to have a well-defined road classification for various reasons, including transferability of benchmarks and comparison of performance indicators. The classification of roads can be based on the various factors, such as, material type, traffic volume, rigidity, surface condition, topography, and location and function.

Primarily, roads can be classified as urban and non-urban roads. Urban roads can be defined as roads located within the boundaries of built-up area or within towns and urban cities. Non-urban roads can be defined as long stretches of roads passing through rural areas and located in open country regions outside the built-up area. The roads can be classified as earthen roads, bituminous roads, concrete roads, gravel roads, etc. based on material type. Based on daily traffic volume, roads can be classified as light, medium and heavy traffic volume roads. Based on rigidity, road pavements can be classified as flexible, semi-rigid and rigid pavements. Further, on the basis of surface condition, roads can also be classified as paved or unpaved roads. Based on topography, roads can be classified as plain area or hilly area roads. The location and functional classification of roads is of significant importance for geometric design of roads.

Functional classification refers to the grouping of roads by the character of service the road provides. The two major considerations taken into account for functional classification of road networks are accessibility and mobility. While mobility refers to the smooth and efficient movement of people and goods, accessibility is the ability to reach opportunities. The functional classification is based on combination of different levels of accessibility and mobility. **Fig. 2.1** shows the functional classification of non-urban roads.

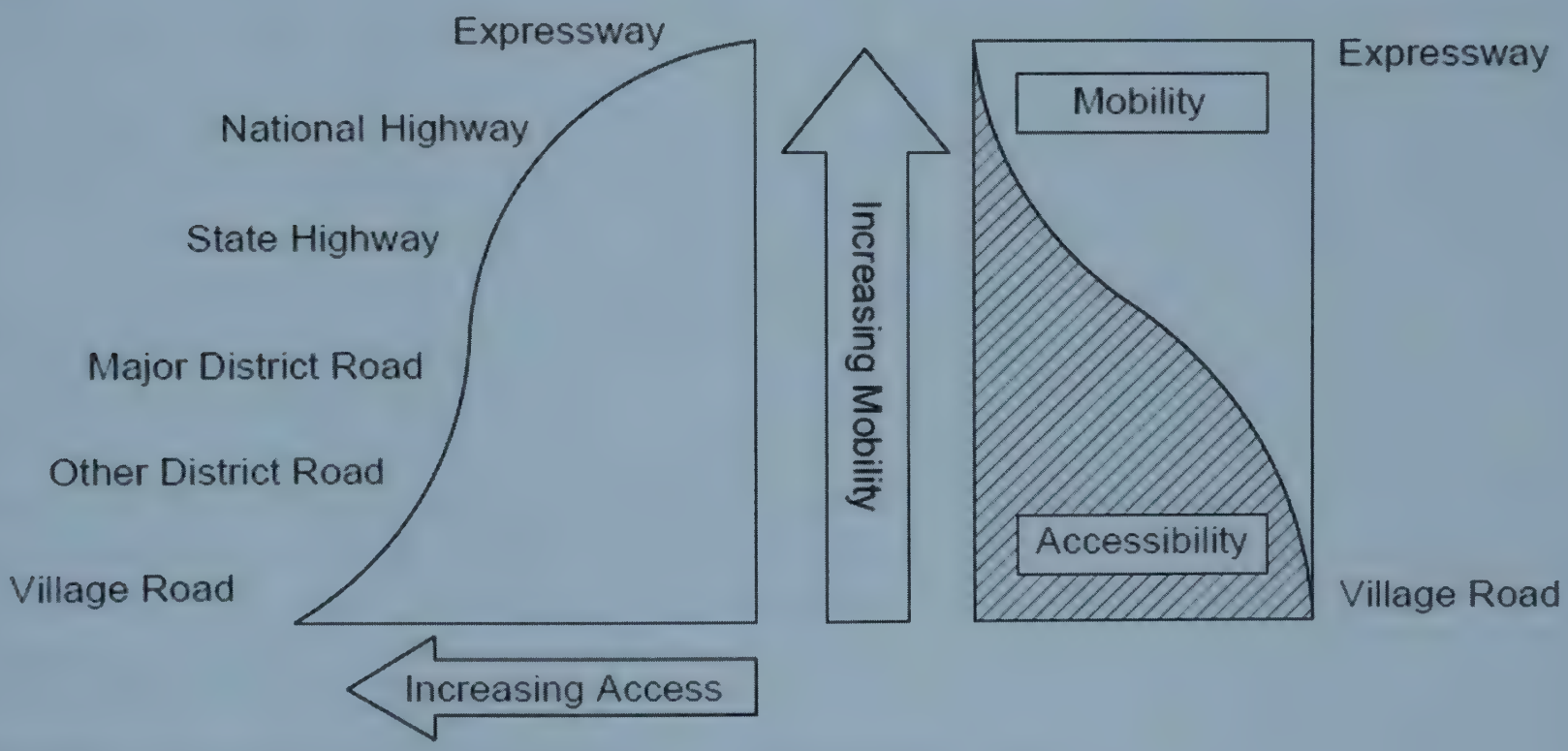


Fig. 2.1 Functional Classification of Non-Urban Roads

2.2 Functional Classification of Non-Urban Roads

Non-urban roads in India are classified into following five categories based on location and function according to Nagpur road plan:

(i) National Highway (NH)

(ii) State Highway (SH)

(iii) Major District Road (MDR)

(iv) Other District Road (ODR)

(v) Village Road (VR)

The present system follows a modified classification system as per third 20-year road development plan. The roads are now classified into following three classes, for the purpose of transport planning, functional identification, earmarking administrative jurisdictions and assigning priorities on a road network:

(i) Primary system
   a. Expressway (EW)
   b. National Highway (NH)

(ii) Secondary system
   a. State Highway (SH)
   b. Major District Road (MDR)

(iii) Tertiary system (Rural Road)
   a. Other District Road (ODR)
   b. Village Road (VR)

The extent and degree of control defines the functional categorization of a road or a highway. The expressways and national highways, having limited access, serve the primary function of mobility, while the function of other district roads and village roads is to provide accessibility instead of mobility. State highways and major district roads play a dual role and are aimed to serve moderate levels of both accessibility and mobility.

Rural roads in India further include single lane roads, intermediate lane roads, two lane roads, and multi-lane highways. Single lane, intermediate lane and two lane roads are undivided roads that facilitate the two-way movement of traffic, whereas multi-lane highways have divided carriageways with two or more lanes in each direction. An undivided road is one which permits traffic flow in both directions without physically segregating the directional movements. A divided road, on the other hand, is the one where the traffic in two directions moves in segregated lanes because of the presence of a physical segregation, such as central verge or median.Traffic operations on two lane or intermediate lane roads are unique in nature. Lane changing and overtaking manoeuvres are possible only in the face of oncoming traffic in the opposing lane. As the traffic volume increases, the overtaking demand increases, while the passing opportunities decline rapidly. Therefore, flow in one direction influences the flow in the other direction. This problem is more acute in the case of mixed traffic where speed differential amongst different vehicle types is significant. It increases the desire for overtaking manoeuvres considerably whereas number of opportunities to overtake is limited. Traffic operations on single lane roads are very much influenced by the condition and width of the shoulders as vehicles are forced to use them during passing or overtaking operations.

2.3 Definitions

This section provides with definitions and additional details on the different classes of non-urban roads.

Expressway

Expressway is a class of highway with superior facilities and design standards, and is meant

as through route catering to a high volume of traffic. Expressway is designed with divided carriageways for high speed travel, with full control of access and provided with grade separators at intersection locations. Generally, only fast-moving vehicles are allowed on expressway. Pedestrians and slow-moving vehicles such as two-wheelers, three-wheelers, bullock carts and tractors are not permitted to ply on expressway. IRC:SP:99 lays down the standards for geometric design and general features for expressways.

National Highway

National highway is the important highway running through the length and breadth of the country connecting major ports, foreign highways, state capitals, large industrial and tourist centres, etc. including roads required for strategic movements for the defence of India. The national highway should be the frame on which the entire road communication should be based. All national highways within India are assigned respective numbers. For example, say NH-16, runs from Chennai to Dankuni near Kolkata covering a distance of 1711 km.

State Highway

State highway is an arterial route of a state linking district headquarters and important cities within the state and connecting them with National Highway or highways of the neighbouring states. This highway serves as the main artery for traffic to and from district roads. At times, a state highway may carry heavier traffic than a national highway. The NH and SH have the same design speed and geometric design specifications. A typical example of state highway, say, SH-1 of West Bengal starts from Bangaon and passes through Chakdaha, Madanpur, Kalyani, Halisahar, Naihati, Ichapore, Barrackpore, Kolkata, Garia, Rajpur, Baruipur, Jaynagar and ends at Kulpi covering a total length of around 151 km.

Major District Road

Major district road is an important road within a district, serving areas of production and markets, and connecting these with each other or with the main highways. The MDR has lower speed and geometric design specifications as compared to highways.

Other District Road

Other district road is a road serving rural areas of production and providing them with outlet to market centres, taluka/tehsil headquarters, block development headquarters, or other main roads.

Village Road

Village road is a road connecting villages or groups of villages with each other and to the nearest road of a higher category.

3. CONTROL FACTORS OF GEOMETRIC DESIGN

3.1 Topography

The geometric design of a roadway is influenced significantly by terrain conditions. Economy dictates choice of different standards for different types of terrain. Terrain is classified by the general slope of the country across the roadway alignment, for which the criteria given in **Table 3.1** should be followed.

Table 3.1 Terrain Classification

S. No.	Terrain Classification	Cross Slope of the Country	
1	Plain	Less than 10 per cent	1 in 10 or more
2	Rolling	10-25 per cent	1 in 10 to 1 in 4
3	Mountainous	25-60 per cent	1 in 4 to 1 in 1.67
4	Steep	Greater than 60 per cent	Less than 1 in 1.67

Note: While deciding the terrain classification, short isolated stretches (say less than 1 km) of varying terrain met with on the road stretch should not be taken into consideration.

3.2 Design Speed

Choice of design speed depends on the functional classification of the road and terrain conditions. It is the basic parameter, which determines all other geometric design features. Design speeds for various classes of roads should be taken as per **Table 3.2**.

Normally "ruling design speed" should be the guiding criterion for correlating the various geometric design features, "minimum design speed" may, however, be adopted in sections where land width, site conditions, including costs, do not permit a design based on the "ruling design speed".

The design speed should preferably be uniform along a given roadway. But, sometimes variations in terrain may make changes in speed unavoidable. Where this is so, it is desirable that the design speed should not be changed abruptly, rather in a gradual manner by introducing successive sections of increasing/decreasing design speed so that the road users get conditioned to the change gradually.

The recommended design speed for the main-line expressway should be 120 km/h. The design speed should be uniform along the expressways. For those sections of expressways where variations in terrain may make changes in speed unavoidable, the design speed shall be changed in the gradual manner, by introducing sections of increasing/decreasing design speed as the case maybe. The recommended design speeds for ramps at service interchange or system interchange and auxiliary lanes should be as per IRC:SP:99.

Table 3.2 Design Speeds for NH/SH/MDR/ODR/VR

S. No.	Nature of Terrain	Percent cross slope of the country	Design Speed (km/h) (4/6/8-lane NH/SH)		Design Speed (km/h) (2-lane NH/SH)		Design Speed (km/h) (MDR)		Design Speed (km/h)(ODR/ VR)	
			Ruling	Minimum	Ruling	Minimum	Ruling	Minimum	Ruling	Minimum
1	Plain	0-10	100	80	100	80	80	65	50	40
2	Rolling	10-25	100	80	100	80	65	50	50	40
3	Mountainous	25-60	60	40	50	40	40	30	30	20
4	Steep	> 60	60	40	40	30	30	20	30	20

Note: 1. For 2/4/6/8 lane highways, wherever service roads are provided, recommended design speed of 40 km/h should be adopted on the service roads.

2. For 2/4/6/8 lane highways, the acceleration and deceleration lanes of service roads should be designed for a speed differential of 60 km/h.

3.3 Other Factors

3.3.1 *Introduction*

Other factors that affect the geometric design of highways are as follows:

- Design Vehicles
- Driver Performance
- Traffic
- Environment and Economy

3.3.2 *Design Vehicles*

The physical characteristics and the proportions of vehicles of various sizes using the highway are key controls in geometric design. Therefore, it is appropriate to examine all vehicle types, establish general class groupings, and select vehicles of representative size within each class for design use. These selected vehicles, with representative weight, dimensions, and operating characteristics (acceleration and deceleration characteristics), which are used to establish highway design controls for accommodating vehicles of designated classes, are known as design vehicles.

The width of the design vehicle has a bearing on the width of traffic lanes and that of shoulders.

The height of the vehicle affects the clearance to be provided in designing road under bridges, electrical service lines, and other overhead structures.

The overall length of the vehicle (including trailer and semi-trailer combinations) has to be taken into consideration in designing horizontal curves and vertical curves, as also in framing safety regulations for passing and overtaking.

In the design of any facility, the designer should consider the largest design vehicle likely to use that facility with considerable frequency or a design vehicle with special characteristics appropriate to a particular intersection in determining the design of critical features such as radii at intersections and radii of turning roadways.

The selection of the vehicle type for design of a road would depend upon terrain conditions, economic justification, importance of the road and similar other considerations. Roads in steep and mountainous terrain need not be designed for truck-trailer combination and may only be designed for single unit vehicle and, where economically feasible, for semi-trailers.

As per MoRTH notification vide G.S.R. 414(E) dated 26th June, 2020, the overall dimensions of motor vehicles shall not exceed the limits as below:

Length

The maximum overall length of a single unit truck (N3 category-vehicle having 2 or more axles) shall be 12 m. The maximum overall length of a single unit bus (M3 with 3 axles including Double Deck) shall be 15 m. The maximum overall length of a truck-trailer/semi-trailer/tractor-trailer combination shall be 18.75 m.

Width

The maximum permissible width of the vehicle can be taken as 2.6 m. However, construction equipment vehicle may have a width upto 3.0 m.

Height

The permissible height for a motor vehicle can be taken as 4.0 m. However, construction equipment vehicle may have a height up to 4.75 m.

Minimum Turning Radii of Design Vehicles

The minimum radius of horizontal curve is governed by the minimum turning radius of a vehicle. The general values are as given below:

(i) Turning circles of public service vehicles should have swept diameters no greater than 20 m for vehicles not exceeding 8.3 m in length; 22 m for vehicles exceeding 8 m in length but not exceeding 11 m in length; and 24 m for vehicles longer than 11 m. The turning circles of commercial vehicles range widely from 9 m to over 26 m in diameter but lie mainly between 12 and 21 m in diameter.

(ii) Radii adequate for commercial vehicles are suitable for private cars. The turning circles of car range between 8 m and 14 m in diameter.

Therefore, while designing sharp curves, one must remember that vehicle cannot be turned from a straight path directly over to full lock and that allowance must be made for the back wheels cutting the corner. A minimum turning diameter of 26 m is therefore, desirable. In mountainous and steep terrain, since the design vehicle is semi-trailer, a minimum turning diameter of 21 m may be accepted.

3.3.3 *Driver Performance*

An appreciation of driver performance is essential to proper roadway design and operation. The suitability of a design rests on how safely and efficiently drivers are able to use the roadway. When drivers use a roadway designed to be compatible with their capabilities and limitations, their performance is aided. When a design is incompatible with the capabilities of drivers, the chance for driver errors increases, which may result in crashes or inefficient operation on roads.

The physiological and psychological characteristics of the human greatly affect the geometric design of the roadway. Vision and hearing are the factors that come under the physiological characteristics.

Vision of a driver is one of the important factors to be considered in the geometric design. The zone of acute vision is formed by a cone whose angle is 3 degrees about the centre of retina. This signifies that the objects should be within this narrow cone for satisfactory perception. However, the vision is still satisfactory when the angle of cone is up to 10 or 12 degrees. This is important when locating traffic signs and signals. Peripheral vision deals with the total visual field for the two eyes. The angle of peripheral vision is about 160 degrees in the horizontal direction and 115 degrees in the vertical direction. Colour vision is important for discerning the traffic lights and colour schemes in traffic signs. The ability of the driver's eyes to adapt to glare due to headlights or to variations in the lighting conditions is an important factor. The ability to judge the height and

distance of an object stereoscopically and its speed are important to the road user. Older drivers have special needs that should be considered in roadway design and traffic control.

Hearing is an aid to the road user which can at times be very vital. The sound of a horn or the sound of the nearby vehicle itself can alert a pedestrian to safely cross or doing other manoeuvre. Elderly persons with poor eyesight can perceive better through hearing than through seeing.

The important psychological characteristics of road user include perception, intellection, emotion and volition, abbreviated as PIEV and the time taken for these processes is known as PIEV time. **Perception** is the process of perceiving the sensations received through the eyes, ears, nervous system and brain. **Intellection** is the identification of the stimuli by the development of new thoughts and ideas. **Emotion** is the personal trait of the individual that governs his decision-making process, after the perception and intellection of the stimuli. **Volition** is the will to react to a situation. This PIEV time is used in the calculation of sight distance. According to AASHTO Green book, average PIEV time ranges from 0.6 seconds to 2 seconds when an event is expected, and it increases by 35 percent in case of unexpected events. Thus, for a simple, unexpected decision and action, some drivers may take as long as 2.7 seconds to respond. A complex decision with several alternatives may take several seconds longer than a simple decision. In India, a design value of 2.5 seconds is taken for calculating the required stopping sight distance and 2.0 seconds for calculating the required overtaking sight distance.

3.3.4 *Traffic*

The volume and characteristics of traffic should be considered for the design of a roadway. Traffic volumes for an interval of time shorter than a day more appropriately reflect the operating conditions that should be used for the design and mostly, in all the cases, adequate time period is considered to be one hour. Due to the changing traffic pattern during the various hours of the day, a key decision is involved in determining the appropriate hourly volumes for design. It would be uneconomical if maximum peak-hour traffic during a year is used for design and if average hourly traffic is used, it would lead to inadequacy. So, always a reasonable value of traffic is considered for the geometric design. The traffic characteristics include directional distribution, composition and speed of traffic which are necessary to be considered for the geometric design.

3.3.5 *Environment and Economy*

The term environment includes human, animal, and plant communities and the forces acting on all the three. The roadway design should be in such a way that it would not affect the sustenance and quality of human life. The design developed considering all the above factors should be economical and must be within the allocated budget for the construction and maintenance of roadways.

The roadway geometric design should be in such a way that the overall aesthetics of the environment is not affected.

4. CROSS-SECTIONAL ELEMENTS

4.1 Right-of-Way

Road land width (also termed the Right-of-Way) is the land acquired for road construction purposes and provision of utilities along the length of road. However, additional land, if required

for accommodating cross sections, improvement of geometrics, realignment, junctions, bypasses etc., should be acquired by the authority. A minimum ROW to be available for development of highways is given in **Table 4.1.** Desirable land widths for other classes of roads are indicated in **Table 4.2.**

Table 4.1 Recommended Right-of-Way for Highways and Expressways

S. No.	Road Classification	Minimum Right of Way
1	2- lane Highways	30 m
2	4- lane Highways	45 m
3	6- lane Highways	60 m
4	8- lane Highways	120 m
5	Expressways	90-120 m
6	2- lane Highways with Bypasses	45-60 m
7	2- lane Highways in Open Areas** (Mountainous and steep terrain)	24 m 18 m (Exceptional)
8	2- lane Highways in Built-up Areas** (Mountainous and steep terrain)	20 m 18 m (Exceptional)

Note: *The ROW width must include the 2 m wide strip on either side reserved for placement of utilities outside the fencing.*

Table 4.2 Recommended Right-of-Way for Other Classes of Roads (in m)

S. No.	Road Classification	Plain and Rolling Terrain				Mountainous and Steep Terrain			
		Open Areas		Built-up Areas		Open Areas**		Built-up Areas**	
		Normal	Range	Normal	Range	Normal	Exceptional	Normal	Exceptional
1	Major District Roads	25	25-30	20	15-25	18	15	15	12
2	Other District Roads	15	15-25	15	15-20	15	12	12	9
3	Village Roads	12	12-18	10	10-15	9	9	9	9

In order to ensure proper sight distance and for the circumstances given below in notes, it will be necessary to acquire additional right of way over that indicated in **Table 4.1** and **Table 4.2

Notes:

1. Right of way shall be enough to ensure minimum setback of 5 m for building line from edge of road land boundary.

2. Additional land is required at locations involving deep cuts to maintain stability of slopes, high fills and unstable/ land slide area.

3. If the road is planned to be upgraded in the future, land width shall correspond to higher class of road.

4. Keeping in view the fast pace of ribbon development of habitation along the roads in hilly/ mountainous region, encumbrance free future expansion and safety of road users, minimum two-lane carriage way road as per national highway standard shall be planned and the land width shall be planned accordingly.

5. In case of village roads where initially it is decided to construct single lane carriage way due to low volume of traffic, the width of the land to be acquired shall be planned for two lane road as per national highway standards in the beginning for safety of road users and to take care of encumbrance free future expansion on account of ribbon development along right of way.

6. Strategic road/ border roads for military/paramilitary/security forces shall be planned for not less than two lane carriage way as per national highway standards and accordingly the land width is to be planned.

7. Wherever, divider/median is required to be provided for the cases of divided highway of all categories on National/ State/District roads, the minimum 1.50-meter-wide divider/median shall be provided and accordingly the land width is to be planned.

The need for a wider right-of-way at important road intersections should be kept in view. **Figs. 4.1** to **4.13** (appended at the end) show the ROW and the typical cross-sections of 2-lane, 4-lane,6-lane, 8-lane highways, and 6-lane, 8-lane expressways under different circumstances.

If a road is expected to be upgraded to a higher classification in the foreseeable future, the land width to be acquired should correspond to the latter. Upgradation of any highway involves, (i) Acquisition of additional land, (ii) Shifting of utilities, (iii) Felling of trees, and (iv) Removal of existing structures which fall in the Right of Way (ROW) of the proposed expansion. Meeting these prerequisites not only entails heavy cost but also involves a lot of time and effort. Further, the felling of grown-up trees leads to adverse effects on environment. Therefore, for upgradation of any national highway, the land for expansion should be acquired on one side of the existing highway. Side of the land acquisition may change from one stretch to another depending upon most optimal alignment. For more details on the expansion of existing highways, refer to "A Manual of Guidelines on Land Acquisition for National Highways under The National Highway Act, 1956".

4.2 Width of Carriageway

The standard width of carriageway for rural highways shall be as indicated in **Table 4.3.** The total width should be determined in relation to the design traffic and capacity of the roadway.

Table 4.3 Width of Carriageway

Type of Facility	Single lane	Multi-lane pavements on Highways, MDR, ODR and VR, width per lane	Multi-lane pavements on Expressways, width per lane
Width (m)	3.75	3.5	3.75

Notes:

1. Except on the important trunk routes, an intermediate carriageway width of 5.5 m may also be adopted instead of regular two lanes if the same is considered advantageous.

2. Expressways should be developed as per six-lane configuration and all the facility planning should be done accordingly. In case, the Authority concludes that a six-lane expressway would be grossly over or under design, the lane configuration could be changed to four or eight-lane respectively in the initial stage only.

3. Where the carriageway width changes, e.g. from single lane to two lanes or two lanes to four lanes or vice-versa, the transition should be affected through a taper of 1 in 15 when it is from narrower to wider and it shall be 1 in 20 when it is from wider to narrower.

4. Where the carriageway width changes from four lanes to six lanes or six lanes to eight lanes or vice-versa, the transition should be affected through a taper of 1 in 50.

5. In hilly areas, minimum two-lane carriageway is recommended and for all types of terrains and all highway classifications, carriageway width in hilly areas for a two-lane road is to be kept as 7.0 m.

4.3 Median Width and Median Openings

4.3.1 *Median Width for 4/6/8 lane Highways*

The median width is the distance between inside edges of individual carriageways of a divided highway segment. The following guidelines should be followed while providing median on 4/6/8 lane highways.

(i) All multi-lane highways shall be provided with depressed/flush median depending upon availability of land. No raised/kerb median shall be provided.

(ii) The minimum width of the depressed median shall be 7.0 m in plain and rolling terrain. In mountainous and steep terrain and in built-up areas of plain and rolling terrain, where there are constraints in terms of availability of land and available median width is 2.5 m or less, flush median could be provided with collapsible crash barrier and anti-glare measures.

(iii) As far as possible, the median shall be of uniform width in a particular section of the highway. However, where changes are unavoidable, a transition of 1 in 50 shall be provided.

(iv) Edge strips, 0.6 m wide, adjacent to the carriageway towards depressed median in either direction shall be paved with same specifications as of the adjoining carriageway.

(v) Metal beam type crash barrier or wire rope safety barrier shall be provided on either side of depressed/flush median.

(vi) The depressed median shall have suitably designed drainage system so that water does not stagnate in the median. IRC:SP:42 shall be referred for more details.

(vii) Suitable shrubs should be provided in case of unpaved median sections (open country sections) as an anti-glare provision and the desirable height of the shrub should not be more than 1.5 m. Collapsible crash barrier arrangement could be provided at suitable intervals for maintenance of shrubs. For more details on shrub plantation and maintenance, refer to IRC:SP:21.

(viii) The median in the built-up areas shall be paved. In such locations, crash barrier/wire ropes and antiglare measures shall be provided for safety consideration and headlight glare respectively. Suitable antiglare measures such as metal/plastic screens shall be provided to reduce headlight glare from opposite traffic. The total height of screen including the height of the barrier shall be 1.5 m, the width of screen is in the range of 80 to 250 mm at a spacing of 0.5 m to 1.0 m.

(ix) In already developed multi-lane highways with raised median, especially in open country/ rural areas, the raised median shall be replaced by depressed/flush median. Alternatively, crash barrier may be provided along raised median in such a way that the face of the crash barriers is flushed with the face of the raised median.

(x) Median width on approach to grade separated structures shall be same as before/end of the approach.

In rolling and hilly country, the median width will be dictated by topography and the individual carriage ways could be at different levels.

4.3.2 *Median Width for Expressways*

The following guidelines should be followed while providing median on expressways:

(i) Only depressed median shall be provided on expressways. No raised median/kerbs shall be provided.

(ii) The desirable width of depressed median shall be 15 m between the outside edge of inside shoulder of the two carriage ways that slope down towards the median center line in 6 H:1 V resulting in a 'V' shaped ditch to act as median drain. For more details, refer to IRC:SP:99.

(iii) An edge strip of 0.75 m width, adjacent to the carriageway towards depressed median in either direction shall be paved with same specifications as of the adjoining carriageway.

(iv) The depressed median shall have suitably designed drainage system so that water does not stagnate in the median.

(v) As far as possible, the median shall be of uniform width in a particular section of the expressway. However, where changes are unavoidable, a transition of 1 in 50 shall be provided.

(vi) Median barriers shall be provided as specified in IRC:SP:99. Suitable antiglare measures such as metal/plastic screens shall be provided to reduce headlight glare from opposite traffic. The total height of screen including the height of the barrier shall be 1.5 m. These shall be located in a manner that the vertical tangent to the thrie beam crests passes through the edge of the paved shoulder.

(vii) To enable full access control facility, thrie beam metal crash barrier shall be provisioned at both edges of roadway and median to ensure safety. These shall be located in such a manner that the vertical tangent to the thrie beam crests pass through the edge of the paved shoulder.

(viii) In case of narrow medians (2 m or less), as generally provided in urban areas, collapsible crash barriers shall be provided along with anti-glare screens on both sides of the median.

(ix) In already developed multi-lane expressway with raised median especially in open country/ rural areas, the raised median shall be replaced by depressed median. Alternatively, crash barrier may be provided along raised median in such a way that the face of the crash barriers is flushed with the face of the raised median.

4.3.3 *Median Openings*

The following criteria should be adopted while providing a median opening on a divided highway segment.

(i) In open country, median openings shall not be spaced closer than 2 km. It shall be ensured that no median openings are provided on horizontal and vertical curve sections.

(ii) In built up area, median opening shall be provided as per site requirement and the spacing between two median openings in built up area shall not be less than 500 m.

(iii) Median opening shall not be provided in front of the service road entry. The distance between the service road entry and the median opening shall be at least equal to the sum of length of acceleration lane or deceleration lane and weaving length. This distance shall however be not less than 150 m. Location of opening shall be so decided as to minimize contraflow.

(iv) All median openings shall be provided with additional 3.5 m wide shelter lane/ storage lane by the side of median in both directions for vehicles waiting to take U-turn. Wherever required, horizontal geometries of the road shall be suitably adjusted.

(v) Length of median opening shall be 18 to 20 m only. Length of median opening can be more than 20 m in case of median opening without storage lane, to serve as neutral place for small vehicles to wait.

(vi) At median openings, all plantations and objects in the median shall be removed and the median shall be paved for the entire length of right turning storage lane including taper or the length equivalent to safe stopping sight distance based on design speed, whichever is higher from the median tip to ensure visibility between the approaching vehicle and the waiting/turning vehicle.

(vii) In case of differential carriage ways at median openings, a maximum level difference of 1 in 10 (desirable) can be allowed with an absolute maximum of 1 in 8 between both the carriage ways.

In case of expressways, there shall be no median opening between any successive ramps causing conflict with the on-coming traffic that needs to be controlled by signals. There shall, however, be a collapsible or movable median barrier at every 5 km interval for traffic management, maintenance works and removal of vehicles involved in accidents. Such barriers shall be located before and after tunnels, interchanges, and rest areas. These barriers shall remain in 'locked' mode, except in case of emergencies. Length of median opening shall not be less than 20 m (for emergency and repair/maintenance works).

4.4 Shoulders

The shoulder width on both sides of the carriageway for all classifications of 2-lane highways, and on outer side of 4/6/8 lane highways should be as given in **Tables 4.4, 4.5** and **4.6. Table 4.4** and **Table 4.5** are applicable for 2-lane and 4/6/8 lane highways respectively in plain and rolling terrain. **Table 4.6** is applicable for highways in mountainous and steep terrain.

In case of highways, where embankment is more than 6 m high, kerb with channel shall be provided at the end of paved shoulder to channelize the drainage as an erosion control device and earthen shoulder shall be raised up to the level of kerb. For embankment slope of 3:1 or flatter, roadside safety barrier is not warranted. For expressways, it is necessary to provide roadside safety barriers on embankments where the recoverable slope up to a distance of clear zone applicable for the design speed is not available. Edge/grooved rumble strips should be provided on highways and expressways at a distance of 0.5 m from the inner edge of the paved shoulder.

For expressways, the shoulder on the outer side (left side of carriageway) shall be 3.0 m wide paved plus 2.0 m wide earthen and 0.75 m wide paved on the inner edge (median side). The shoulder composition shall be as below:

(i) The composition and specification of the paved shoulder shall be as that of the main carriageway.

(ii) The earthen shoulder shall be provided with 200 mm thick layer of non-erodible/granular material (preferably fine graded GSB material) for protection against erosion.

In case of MDR/ODR/VR, the minimum shoulder width on each side should be 2.0 m (0.5 m paved and 1.5 m unpaved) in plain and rolling terrain. In mountainous and steep terrain, the minimum shoulder width on each side should be 1.5 m (0.5 m paved and 1.0 m earthen). In any case, the total minimum roadway width (carriageway and shoulders on both sides) should not be less than 7.0 m for MDR/ODR/VR.

Table 4.4 Width of Shoulders in Plain and Rolling Terrain (2-Lane Highways)

Type of Section	Width of Shoulder (m) on either side		
	Paved	Earthen	Total
Open country with isolated built up area	1.5	1.0	2.5
Built up area (2-lane section)	2.5	-	2.5
Built up area (4-lane section)*	-	-	-
Approaches to grade separated structures	1.5	-	1.5
Approaches to bridges	1.5	1.0	2.5

**(The stretch in built up area shall be provided with facilities such as footpath etc. for vulnerable road users to segregate them)*

Table 4.5 Width of Shoulders in Plain and Rolling Terrain (4/6/8 Lane Highways)

Type of Section	Width of Shoulder (m) on either side		
	Paved	Earthen	Total
Open country with isolated built up area	2.0	1.5	3.5
Built up area	2.0	-	2.0
Approaches to grade separated structures	2.0	-	2.0
Approaches to bridges	2.0	1.5	3.5

Table 4.6 Width of Shoulders in Mountainous and Steep Terrain (Hilly Area)

Type of Section		Width of Shoulder (m)		
		Paved	Earthen	Total
Open country with isolated built up area	Hill Side	1.5 m	-	1.5 m
	Valley Side	1.5 m	1.0 m	2.5 m
Built up area and approaches to grade separated structures/ bridges	Hill Side	0.25 m +1.5 m (Raised)	-	1.75 m
	Valley Side	0.25 m +1.5 m (Raised)	-	1.75 m

Notes:

1. Cross section in the approaches to grade separated structures should be continued up to 50 m length before providing the taper to enter in the next cross section to provide driver comfort due to sudden change of cross section. It will also help in merging of service road. Further, cross section of structure and approaches should be same or smooth taper should be provided as per the requirements.

2. Due to frequent cuts during rainfall and for provision of metal beam crash barrier, it may be desirable to have earthen shoulder of width of 2 m on valley side. It may also be desirable to provide an earthen shoulder of 1 m on hill side.

3. In case of hilly areas, strategic road/ border roads for military/ paramilitary/ security forces shall be constructed for not less than two lane carriageway along with paved shoulder on hill side + paved and earthen shoulder on valley side on same lines of National Highway (IRC:SP:48).

4.5 Roadway Width

The width of roadway shall depend upon sum of the widths of carriageway, shoulders and the median. The minimum and maximum values of the roadway elements under different conditions have been discussed in the respective sections and accordingly roadway width could be calculated. **Figs. 4.1** to **4.13** can be used as a reference for determining roadway width for typical sections of 2/4/6/8 lane highways and 6/8 lane expressways.

4.5.1 *Passing Places for Roads in Mountainous and Steep Terrain*

Passing places or lay-byes should be provided on single lane roads in mountainous and steep terrain to cater to the following requirements:

a) To facilitate crossing of vehicles approaching from opposite direction

b) To tow aside an out-of-order vehicle so that it does not obstruct the main stream traffic.

Normally the passing places/lay-byes should be 3.75 m wide, 30 m long on the inside edge (i.e. towards the hill side), and 20 m long on the valley side.

The exact location of passing places shall be judiciously determined taking into consideration the available extra width and visibility. In general, passing places should be provided at the rate of 2-3 numbers per kilometer.

4.6 Roadway Width for Cross-Drainage Structures

4.6.1 *General*

Cross-drainage structures are difficult to widen at a later stage. As such, the roadway width for them should be decided very carefully at the planning stage itself. For roads being built to lower standards initially for some reasons, or those which are expected to be upgraded/widened in the foreseeable future, it will be desirable to go in for a higher roadway width at the cross-drainage structures right in the beginning. It is suggested to keep roadway width of culverts/ structures equal to roadway width of approach road. This practice is being followed to avoid vehicle crashes with parapets.

4.6.2 *Culverts*

In plain and rolling terrain, the overall width on culverts (measured from outside to outside of the parapet walls) should be equal to the normal roadway width on approach. In mountainous and steep terrain, the clear roadway width available on the culverts (measured from inside to inside of parapet walls or kerbs) should be calculated based on the widths of the corresponding roadway elements.

For high embankments, it is recommended to provide minimum one balancing culvert @ 1 km interval (if no culvert exists) on the highway. In the likelihood of building up of water due to high intensity rainfall or sudden release of water from an u/s dam/weir on the stream, a pair of relief culverts on either side of existing culvert or bridge (say at 200 m distance) shall be provided. Such culverts are planned considering topography and rainfall in the region such that new-channels do not form, but permit excess discharge to rejoin the existing stream at a minimum distance of 100 m on the d/s side. Every effort shall be made by the Highway Agency to coordinate with Irrigation Department in the planning and construction stage itself.

4.6.3 *Bridges and Causeways*

At bridges and causeways, the clear width of roadway between kerbs should be more than or equal to the width of the approaching roadway in any case.

Where a footpath is provided for the use of pedestrians, its width on each side should not be less than 1.8 m (desirable) with an absolute minimum width of 1.5 m.

4.7 Pavement Camber/Cross fall

The cross fall on straight sections of road carriageway, paved shoulders and paved portion of median shall be 2.5 percent for bituminous surface and 2.0 percent for cement concrete surface for all classes of roads.

For 2/4/6/8 lane highways, the cross fall for earthen shoulders on straight portions shall be at least 0.5 percent and desirably 1.0 percent steeper than the slope of the pavement and paved shoulder. In case of expressways, the cross fall for earthen/granular shoulders on straight portions shall be at least 1.0 percent steeper than the slope of the pavement and paved shoulder. On super elevated sections of highways and expressways, the earthen portion of the shoulder on the outer side of the curve shall be provided with reverse cross fall so that the earth does not drain on the carriageway and the storm water drains out with minimum travel path. The reverse crossfall shall be taken as 0.5 percent for highways. The maximum roll over angle in case of super elevated sections may be considered as 7.5%. If the super elevation is less than 7%, for example 5%, the shoulder cross fall can be kept as maximum of 2.5% for effective drainage of shoulder. The cross fall of right turning storage lanes shall be 0.5 percent in the direction of super elevation to restrict reverse slope in the median opening.

In case of 4/6/8 lane highways, the cross fall shall be unidirectional for either side of the carriageway sloping towards the shoulder in straight reaches and towards the lower edge on horizontal curves. The camber on the existing road shall be modified to unidirectional cross fall. For expressways also, each carriageway shall have unidirectional cross fall. In case of multi-lane highways and expressways, to avoid storm water on outer carriageway to flow onto adjacent carriageway, there shall be longitudinal and cross drains in the median to drain the same. There shall not be median cuts, as stated in IRC:SP:42. This is unsafe as there is chance of aquaplaning at high speeds.

Generally, undivided roads on straight sections should be provided with a crown in the middle and surface on either side sloping towards the edge. However, on hill roads this may not be possible in every situation, particularly in reaches with a winding alignment where straight sections are few and far between. In such cases, discretion may be exercised, and, instead of normal camber, the carriageway may be given a uni-directional cross fall towards the hill-side having regard to factors such as the direction of super-elevation at the flanking horizontal curves, ease of drainage, problem of erosion of the down-hill face, etc.

4.8 Service Roads and Acceleration/Deceleration Lanes

Service roads are provided to control the haphazard access of road users to the high-speed facilities, mostly near the built-up areas, and to make the movement of slow-moving vehicles safe and streamlined without hampering the flow of the traffic along the main carriageway. Service roads are generally provided along highways and must be accompanied by properly designed deceleration lane at the beginning of the service road and acceleration lane at the end of the service road for safe and convenient entry and exit of the vehicles from and to the main highway.

4.8.1 *2-lane Highways*

The width of service roads should be generally 7 m but in any case, not less than 5.5 m depending on the availability of ROW. A minimum design speed of 40 kmph shall be adopted for service

roads. Length of the acceleration/deceleration lane* shall be designed for a speed differential of 60 kmph. A minimum width of 5.5 m must be provided. Taper by 1 in 15 beyond the design length at the merging point should be provided. (*__Note__: *The length of acceleration/deceleration lanes should be considered separately from that of the service road.*)

4.8.2 *4-lane Highways*

The width of service roads in open country with isolated built up area should be 7 m with 1.5 m earthen shoulder. In built up area, the service road width shall be 7.5 m (including kerb shyness of 0.25 m on either side) with raised footpath/separator and for more details IRC:SP:84 can be referred.

For the stretches with bridge length less than 60 m where service road is required to be provided on both sides of the stream, then the service road shall continue across the stream and suitably designed 2-lane bridge structures shall be provided. In case, the length of bridge exceeds 60 m, separate bridge structure for service road may not be provided and the service road shall be merged with the highway at 50 m distance before the bridge approach.

A minimum design speed of 40 kmph shall be adopted for service roads. Length of the acceleration/ deceleration lane* shall be designed for a speed differential of 60 kmph. A minimum width of 5.5 m must be provided. Taper by 1 in 15 beyond the design length at the merging point should be provided.(*__Note:__ *The length of acceleration/deceleration lanes should be considered separately from that of the service road.*)

4.8.3 *6-lane and 8-lane Highways*

The width of service roads in open country with isolated built up area should be 7 m with 1.5 m earthen shoulder. In built up area, where separator is provided between main carriageway and service road and RCC/Cement Concrete lined drain-cum-footpath on ROW side are provided, no earthen shoulder shall be provided for service road. The minimum service road width shall be 7 m (excluding kerb shyness of 0.25 m on either side) with raised footpath/separator and for more details IRC:SP:87 can be referred.

For the stretches with bridge length less than 60 m where service road is required to be provided on both sides of the streams, then the service road shall continue across the stream and suitably designed 2-lane bridge structures shall be provided. In case, the length of bridge exceeds 60 m, separate bridge structure for service road may not be provided and the service road shall be merged with the highway at 50 m distance before the bridge approach.

A minimum design speed of 40 kmph shall be adopted for service roads. Length of the acceleration/ deceleration lane* shall be designed for a speed differential of 60 kmph. A minimum width of 5.5 m must be provided. Taper by 1 in 15 beyond the design length at the merging point should be provided.(*__Note:__ *The length of acceleration/deceleration lanes should be considered separately from that of the service road.*)

4.8.4 *Expressways*

Expressways are fully access controlled and thus interchanges with ramps are provided on them. No service roads shall be provided within the ROW of the expressway. Each of the entry and exit ramps must have acceleration/deceleration lane to/from the expressway. The length

shall be determined based on the speed differentials between expressway and ramp speeds. For the minimum length of acceleration/deceleration lanes and other details refer to IRC:SP:99.

5. SIGHT DISTANCE

5.1 Introduction

Visibility is an important requirement for safe and efficient operation of vehicles on a roadway. Therefore, in order to ensure greater safety in the roadway alignments, the design must ensure that the sight distance at every section of the road is of adequate length to permit drivers enough time and distance to control their vehicles in different situations so as to avoid unforeseen and unwarranted incidents. In addition, the sight distance should be sufficient to provide ample opportunities for a vehicle to overtake slow moving vehicles on the road without the risk of collision with the opposing side vehicles. It may be mentioned that provision of adequate overtaking sight distance is a matter of utmost importance along the entire length of two-lane undivided national highways and state highways.

Along with design speed, sight distance also is an important aspect regarding design of an alignment for the given roadway section. Three types of sight distance are relevant and considered in the design:

- Stopping Sight Distance;
- Overtaking Sight Distance; and
- Intermediate Sight Distance

Stopping Sight Distance (SSD) is the minimum sight distance for which all roads must always be designed, regardless of any other consideration. It is the clear distance ahead needed by a driver to stop the vehicle before meeting a stationary object in vehicle's path on the road or any obstruction for which the driver may need to stop the vehicle. In simple words, it is the minimum length of the road which should be visible to the driver at all times.

Overtaking Sight Distance (OSD) is the minimum sight distance on a roadway required by a vehicle/driver to overtake a slow-moving vehicle ahead i.e. travelling in the same direction safely against approaching traffic in the opposite direction.

Intermediate Sight Distance (ISD) or desirable minimum sight distance refers to sight distance provided at sections of roads where the customary overtaking sight distance cannot be provided; and therefore such sections should be designed, as far as possible, for intermediate sight distance.

In general, sight distance is dependent on several factors, such as, the speed of the vehicle, perception-reaction time, coefficient of road friction, gradient, acceleration and deceleration of vehicles, vehicular characteristics and speed of the vehicle moving in opposite lane. Even after considering the above factors, the required sight distance may vary across drivers. This is due to the variations in the ability of the driver to control the vehicle, training, experience, etc.

While applying the designed sight distance standards, efforts should not be concentrated to limit the design of any roadway to the minimum values laid down. Where conditions are favorable,

adopting more liberal values, particularly for stopping sight distance, will be a good engineering practice.

The standards laid down for sight distance in this section are applicable for single lane, intermediate lane, two-lane, multi-lane divided roadway segments and expressways. The standards suggested are uniform for 2/4/6/8 lane highways i.e., the standards for stopping sight distance and intermediate sight distance are given in such a way that there is no problem in upgradation of 2-lane highways to 4-lane highways, 4-lane/6-lane highways to 6-lane/8-lane highways.

5.2 Criteria for Measuring Sight Distance

5.2.1 *General*

Sight distance is that distance along a roadway up to which an object of particular height is constantly visible to the driver. This distance depends on the height of the driver's eye, the height of the specified object above the road surface, and also the height and lateral position of sight obstructions within the driver's line of sight. The criterion for measurement of stopping sight distance is based on the car operation and does not consider the operation of heavy vehicles. The heavy vehicles, in general, require longer sight distances to stop the vehicle. However, this consideration is balanced by the fact that heavy vehicle drivers are able to see much farther beyond the obstructions due to the higher position of the seat in vehicle. Therefore, separate sight distances are not considered for heavy vehicles and passenger cars.

5.2.2 *Height of Driver's Eye*

For all sight distance calculations, the height of the driver's eye is considered to be 1.2 m above the road surface. This value is based on the average vehicle heights.

5.2.3 *Height of Object*

For stopping sight distance, the height of object is considered to be 0.15 m above the road surface. The height of object is considered to be 1.2 m above the road surface for intermediate sight distance and overtaking sight distance calculations. The selection of height of object for stopping sight distance calculations is based on the average height of the object below which there would be no crash risk to the drivers. For overtaking sight distance calculations, the height of the object is taken as the same as the height of the driver's eye since the object is also another vehicle and the drivers of both the vehicles should be able to see each other. Criteria for measuring different types of sight distance discussed above are given in **Table 5.1**.

Table 5.1 Criteria for Measuring Sight Distance

Sl. No.	Sight Distance	Driver's Eye Height	Height of Object
1	Safe stopping sight distance	1.2 m	0.15 m
2	Intermediate sight distance	1.2 m	1.2 m
3	Overtaking sight distance	1.2 m	1.2 m

5.2.4 *Sight Obstructions*

On a tangent roadway, the obstruction that limits the driver's sight distance is the road surface at some point on a crest of a vertical curve. In case of horizontal curves, the driver's sight distance may be limited due to some point on a crest of vertical curve or it may be due to some physical feature outside of the travelled way, any kind of structure, median, median plantation, kerb, a longitudinal barrier, a bridge-approach fill slope, a tree, or the back slope of a cut section. Sight obstructions along roadside and inside the median due to shrubs and tree plantation must be checked as per guidelines provided in IRC:SP:21 and IRC:SP:103 as well as respective specifications provided in IRC:SP:84 for 4-lane divided roads and in IRC:SP:87 for 6-lane divided roads. Therefore, the highway construction plans and drawings should be checked in both the vertical and horizontal planes for limitation to sight distance due to obstructions such that the sight lines should fall within the toe line of the earthworks or within the earthen shoulder. **Fig. 5.1** shows a typical example of obstruction to visibility at horizontal curves.

5.3 Stopping Sight Distance

5.3.1 *Design Criteria*

In general, it must be ensured that the length of the stopping sight distance at every section of the road should be greater than or equal to the minimum sight distance required for stopping the vehicle.

Stopping sight distance has two components:

- The distance travelled during perception and brake reaction time; and
- The distance travelled during the braking time till the vehicle comes to a stop.

Minimum stopping sight distance is the sum of the above two components.

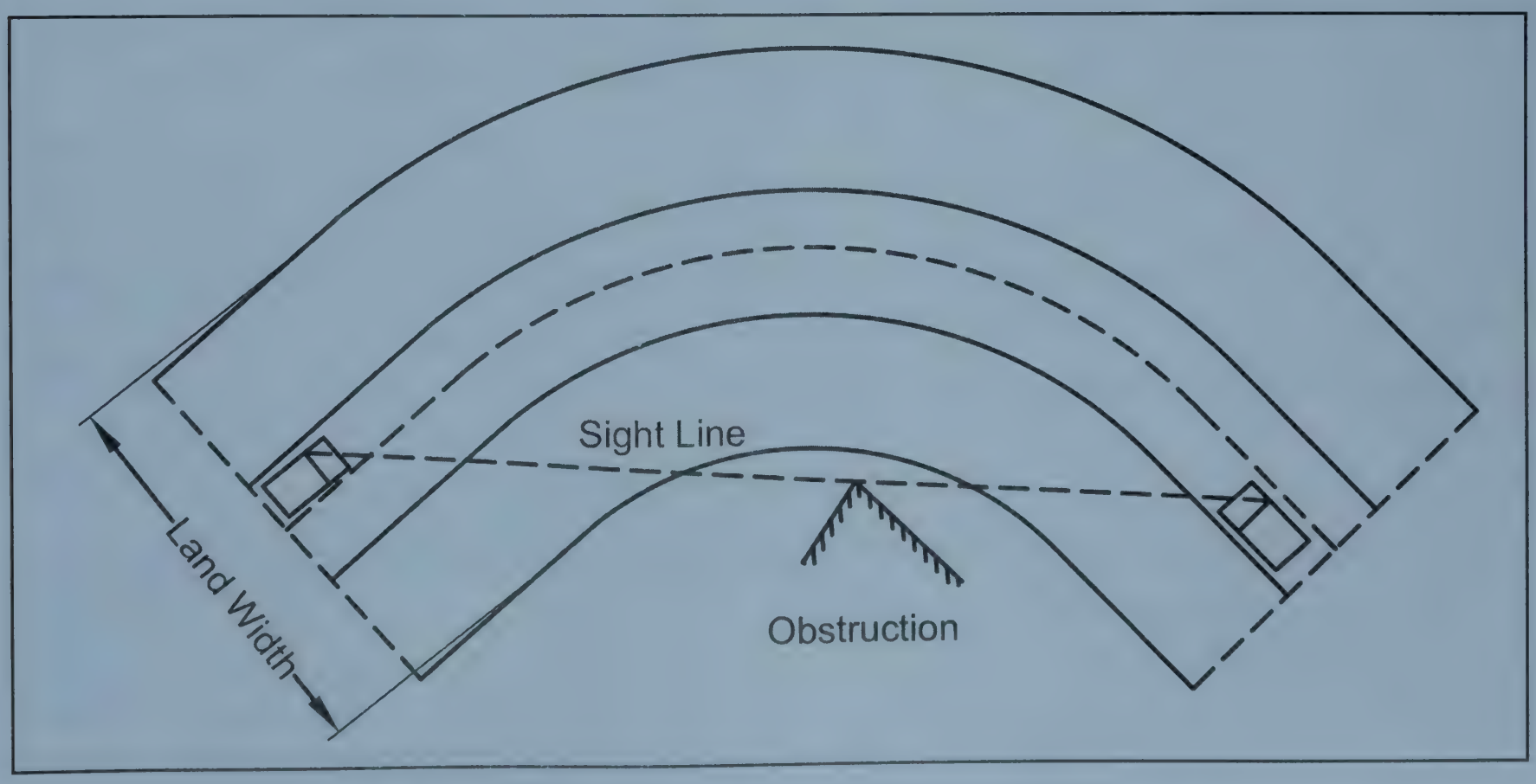


Fig. 5.1 Obstruction to Visibility at Horizontal Curve

5.3.2 *Perception and Brake Reaction Time*

Perception and brake reaction time is the time interval between the instant the driver sights a dangerous object for which a stop is necessary and the instant the brakes are applied. The driver should be clearly able to see the stationary object or a slow-moving vehicle/ pedestrian and perceive it as a danger against the backdrop of the roadway, trees, electric lines, road boundaries, walls, bridges, etc. It takes a certain amount of time to assess the danger, to take the desired action of braking or to slow down the vehicle.

Perception and brake reaction time depends on a variety of factors, viz., age, gender, alertness and visual acuity of the driver, atmospheric visibility, vehicle design, the size and type of the object, etc. Therefore, the total time taken to respond varies across the drivers. The same driver may have different reaction times in a simple situation, such as driving along straight section of a road, as compared to complex driving situations, such as driving on curves, driving in rain, fog, etc. For the purpose of highway design, the total reaction time should be large enough to cover nearly all drivers and highway conditions. A value of 2.5 second is considered reasonable for most situations. The distance travelled during this time is given by the following expression:

$$d_1 = 0.278Vt \qquad \ldots (5.1)$$

where d_1 = distance travelled during total reaction time in meters; V=speed in km/h; and t = the perception and brake reaction time in seconds

The **equation 5.1** in S.I. units can be expressed as:

$$d_1 = vt \qquad \ldots (5.2)$$

where, v= speed in m/s and t = the perception and brake reaction time in seconds

5.3.3 *Braking Distance*

Braking distance is the distance required for a vehicle to come to stop after the brakes are applied. On a level road, assuming friction remains constant during the period of deceleration, braking distance is given by:

$$d_2 = \frac{V^2}{254f} \qquad \ldots (5.3)$$

where d_2 = braking distance in meters; V=speed in km/h; and f= coefficient of longitudinal friction between vehicle tyres and road pavement

The **equation 5.3** in S.I. units can be expressed as:

$$d_2 = \frac{v^2}{2gf} \qquad \ldots (5.4)$$

where, v=speed in m/s; and g= acceleration due to gravity (usually taken as 9.81 m/s^2)

The value of the coefficient of friction varies with speed, tyre pressure, condition of tyre tread, type and condition of pavement, and whether the surface is wet or dry. For design purposes, the value should encompass nearly all significant pavement surface types and field conditions, and should be safe for tyres in reasonable condition. Based on these considerations, design values for coefficient of friction at different vehicle speeds are given in **Table 5.2**.

5.3.4 *Design Values*

Minimum stopping sight distance is given by the sum of the components d_1 and d_2 discussed in the preceding paragraphs, which can be rewritten as

$$d_{SSD} = d_1 + d_2 \quad \text{... (5.5)}$$

$$d_{SSD} = 0.278\, Vt + \frac{v^2}{254f} \quad \text{... (5.6)}$$

The **equation 5.6** in S.I. units can be expressed as:

$$d_{SSD} = vt + \frac{v^2}{2gf} \quad \text{... (5.7)}$$

where, d_{SSD}= Minimum stopping sight distance required

Estimated values of safe stopping sight distance for different design speeds are given in **Table 5.2**. In addition, the desirable minimum sight distance which is equivalent to twice the safe stopping sight distance is also provided in **Table 5.2**. These values have been suggested based on perception and brake-reaction time of 2.5 seconds and coefficient of longitudinal friction varying from 0.40 at 20 km/h to 0.35 at 100 km/h.

The requisite sight distance should be available across the inner side of horizontal curves. In case, where horizontal and summit curves overlap, the required sight distance should be available in both horizontal direction i.e., along the inner side of the curve and vertical direction i.e., along the pavement as shown in **Fig. 5.2**.

Table 5.2 Stopping Sight Distance for various Speeds

Design Speed	**Perception and brake reaction**		**Braking**		**Safe stopping sight distance (m)**		**Desirable minimum sight distance (m)**
V (km/h)	Time, t (sec)	Distance (m) $d_1=0.278Vt$	Coeff. of longitudinal friction (f)	Distance (m) $d_2=\frac{V^2}{254f}$	Calculated Values (m) d_1+d_2	Design Values (m)	
20	2.5	14	0.40	4	18	20	40
25	2.5	17	0.40	6	24	25	50
30	2.5	21	0.40	9	30	30	60
40	2.5	28	0.38	17	44	45	90
50	2.5	35	0.37	27	61	60	120
60	2.5	42	0.36	39	81	80	160
65	2.5	45	0.36	46	91	90	180
80	2.5	56	0.35	72	128	130	260
100	2.5	70	0.35	112	182	180	360
120	2.5	83	0.35	162	246	250	500

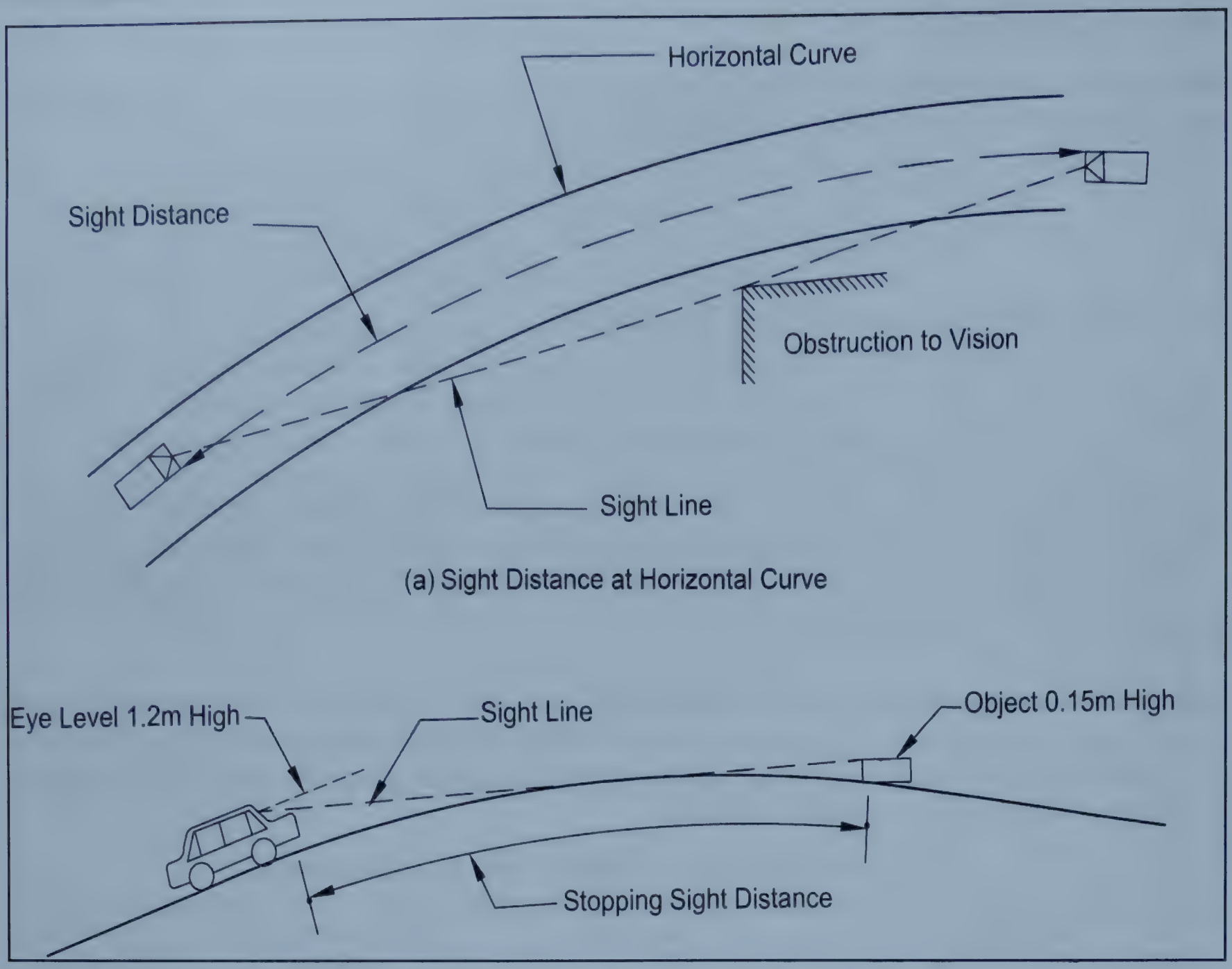


Fig. 5.2 Stopping Sight Distance at Horizontal and Vertical Curves

5.3.5 *Effect of Grade*

The gradient of road is encountered while travelling in both vertical directions. However, the sight distance at a grade section of the road is different in each direction. The braking distance of a vehicle is longer on downgrades and shorter on upgrades. Therefore, the sight distance required on downgrade is larger than on upgrade.

The braking distance formula amended to take the effect of grade into account is:

$$d_2 = \frac{V^2}{254\,(f \pm 0.01G)} \qquad \ldots (5.8)$$

in which G is the longitudinal grade in percent (positive for upgrade and negative for downgrade) and other terms are the same as previously defined.

For example, for a two-lane road with a design speed of 50 km/h and 3% grade, d_2 for vehicle travelling the downgrade will be 29 m and vehicle travelling upgrade d_2 will be 25 m. Correction for grade must invariably be considered for roads with one-way traffic and divided highways which have independently designed profiles.

5.4 Overtaking Sight Distance

5.4.1 *Design Criteria*

The majority of the rural roads in India are two lane highways and undivided roads. For attaining a higher level of service on undivided roads, it is necessary that vehicles moving at design speed should be able to frequently overtake vehicles slower than them. Since overtaking maneuver involves the occupation of road space normally used by opposing traffic, drivers must have sufficient sight distance available to them on a two-way road so that the whole operation can be accomplished with safety. Optimum condition for design is the one in which the overtaking driver can follow the vehicle ahead for a short time while the driver assesses the chances of overtaking, pull out, overtake, and return to the left-hand side of the road before meeting any oncoming vehicle travelling from the opposite direction at the design speed.

In actual practice, there may be occasions to consider multiple overtaking where two or more vehicles overtake another vehicle, or are themselves overtaken in a single maneuver. It is, however, not practical to assume such conditions in developing minimum sight distance criteria. Sight distance values recommended here pertain basically to overtaking maneuvers involving single vehicle. Longer sight distances may generally be available along certain road alignments, e.g., in long relatively level sections, where occasional multiple overtaking can take place without difficulty.

The overtaking sight distance is always more than the stopping sight distance on a two-lane highway. The actual required overtaking sight distance changes according to the driver behavior of the overtaking vehicle, overtaken vehicle and oncoming vehicle. However, for computing minimum overtaking sight distance, certain assumptions for traffic behavior are necessary. The assumptions made are:

- The vehicle being overtaken is travelling at a uniform speed which is 16 km/h less than the design speed of the road;
- The overtaking vehicle follows the vehicle ahead for a short while to perceive the clear road ahead before beginning the overtaking movement;
- Overtaking is done by accelerating rapidly to the design speed and is regarded as having been completed when the overtaking vehicle returns to its own side of the road;
- Overtaking once begun is finished in the face of an oncoming vehicle travelling at design speed in such a way that the latter arrives alongside the former just at the completion of the maneuver.

Fig. 5.3 represents the overtaking maneuver where the vehicle A is in the process of overtaking slow moving vehicle B, while vehicle C is approaching in the opposite lane at the design speed. The entire overtaking operation can be divided into three parts (d_1, d_2, d_3) as follows:

(1) d_1 is the distance travelled by vehicle A during reaction time 't' sec of the driver from position A_1 to A_2

(2) d_2 is the distance travelled by vehicle A, from position A_2 to A_3 during the actual overtaking operation in time 'T' seconds

(3) d_3 is the distance travelled by opposite lane vehicle C from position C_1 to C_2 during the actual overtaking operation of A, i.e. 'T' seconds

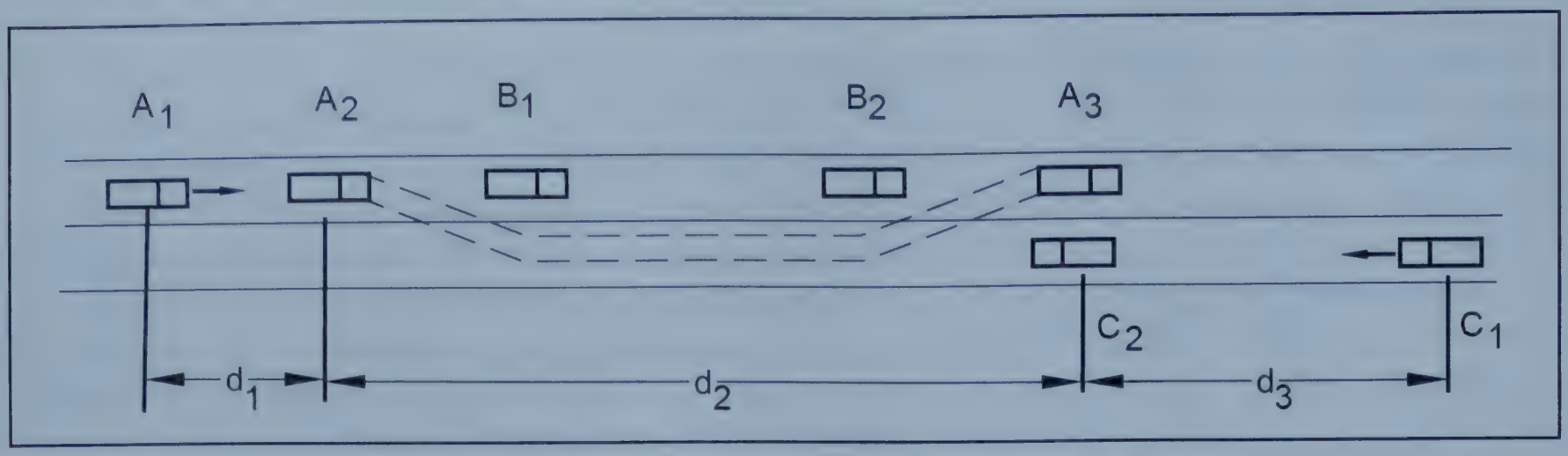


Fig. 5.3 Overtaking Maneuver

Observations in U.S.A. and elsewhere have shown that the overtaking maneuver takes roughly 8 to 14 seconds for a vehicle travelling at the design speed. The distance travelled by an opposing vehicle during the time of overtaking maneuver should be added to this to minimize the chance of a head-on collision while the overtaking vehicle is on the right-hand side of the carriageway. Conversely, this distance should be the distance traversed by an opposing vehicle during the entire time of the overtaking maneuver. But this makes the overtaking distance too long and proper engineering decisions are necessary. During the first phase of the overtaking maneuver, when the overtaking vehicle has not yet pulled abreast of the vehicle being overtaken, the former can always return to its own side if an oncoming vehicle is sighted. The interval of the first phase maneuver is about one-third the total time required for overtaking. On this basis, the element in the overtaking sight distance for the opposing vehicle can be reasonably taken to be the distance it traverses during two-thirds of the actual time for overtaking. The opposing vehicle is assumed to travel at design speed during this period.

The overtaking sight distance should be provided on two-lane highways as much as possible. Due to limitations of cost, design speed of highway as well as topography of the area, the overtaking sections may be limited. In such cases, the drivers should be informed well-in-advance not to perform any overtaking maneuver as it may lead to severe road accidents.

5.4.2 *Design Values*

Using the above assumptions, design values for safe overtaking sight distance for different speeds are given in **Table 5.3.** These are based on a time component of 8 to 14 seconds for the actual overtaking maneuver depending on design speed, increased by about 2/3rd to take into account the distance travelled by a vehicle from the opposite direction during the same time. Generally, for the most of the situations in field, **Table 5.3** can be used to provide the sufficient overtaking distance.

Table 5.3 Overtaking Sight Distance for different Speeds

Speed (km/h)	Time Component (s)			Safe Overtaking Sight Distance* (m)
	For Overtaking Maneuver	For Opposing Vehicle	Total	
40	9	6	15	165
50	10	7	17	235
60	10.8	7.2	18	300
65	11.5	7.5	19	340
80	12.5	8.5	21	470
100	14	9	23	640

** Rounded off values*

The design values in **Table 5.3** pertain to overtaking of a vehicle by a passenger car at level grade. On upgrades, the sight distance required would be more due to reduced acceleration of the overtaking vehicle and the likely speeding up of the vehicle from opposing direction. These factors are somewhat compensated by the loss in speed of the overtaken vehicle which may frequently be a heavy truck. No separate design values are, therefore, recommended for application on grades. **Fig. 5.4** shows the overtaking sight distance criteria on an upgrade.

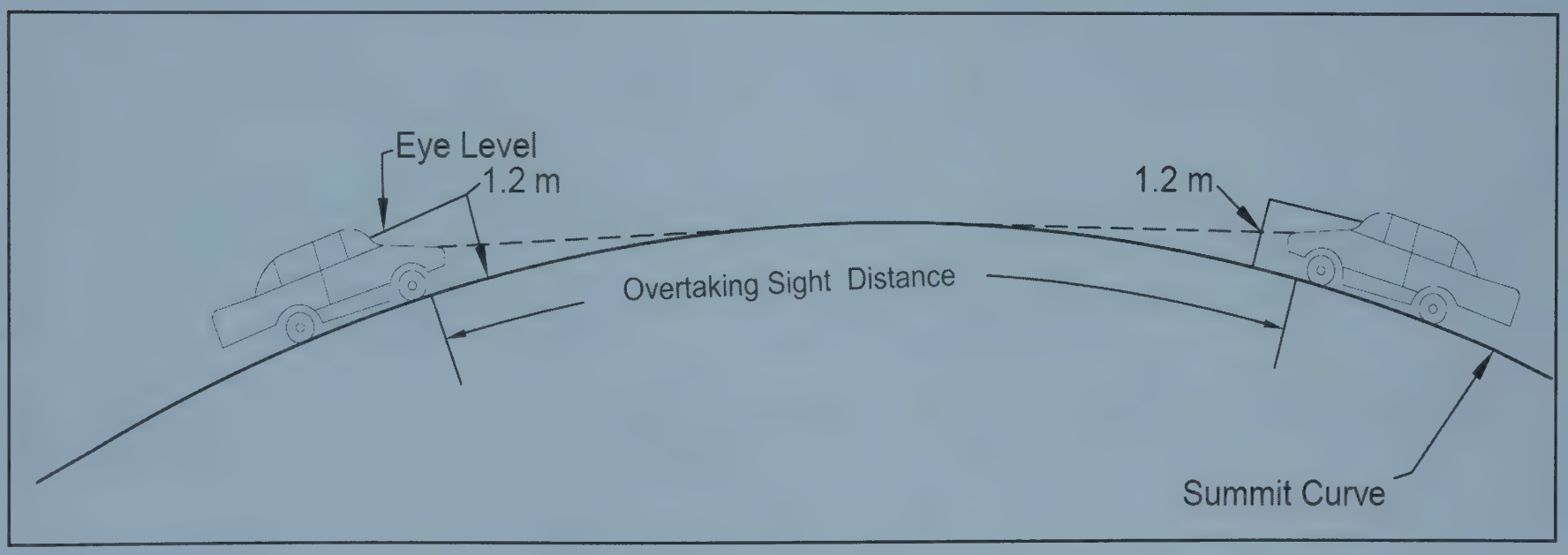


Fig. 5.4 Overtaking Sight Distance on Upgrades

5.4.3 *Provision of Overtaking Zones*

It is advantageous to construct highways in such a way that length of road ahead is visible and sufficient for overtaking. However, this cannot be ensured at every point or section. There may be stretches where safe overtaking distance cannot be provided or visibility is restricted and therefore, overtaking is dangerous. Within such stretches of the highway, the road sign should be erected at the beginning and at intervals within, indicating, "No overtaking" or "Overtaking Prohibited". The "No Overtaking" sign should be erected on each side of the road at the start of the affected length and should be supplemented by repeater signs at intervals not exceeding 400 m. For further details on signs, IRC:67 can be referred.

The opportunity for overtaking must be provided for vehicles moving at design speed at frequent intervals or wherever possible. These zones, which are meant for overtaking, are called as

overtaking zones. The frequency and length of such zones depends on the topography, the design speed of the highway and the cost.

The overtaking sight distance and road width should be sufficient for safe overtaking operations. At overtaking sections, the minimum overtaking sight distance should be $d_1+d_2+d_3$ on a two-lane road in both directions. As no traffic is expected to encroach the right of way from the opposite direction on divided highways and one-way traffic roads, the sight distance should be provided such that vehicle is able to stop safely.

Fig. 5.5 shows the provision of an overtaking zone on a highway. Appropriate sign posts should be provided at sufficient distance in advance to indicate the start and end of overtaking zones. It is desirable to have the length of the overtaking zones equal to five times the overtaking sight distance. If that is not possible, the minimum length of overtaking zone should be three times the safe overtaking distance.

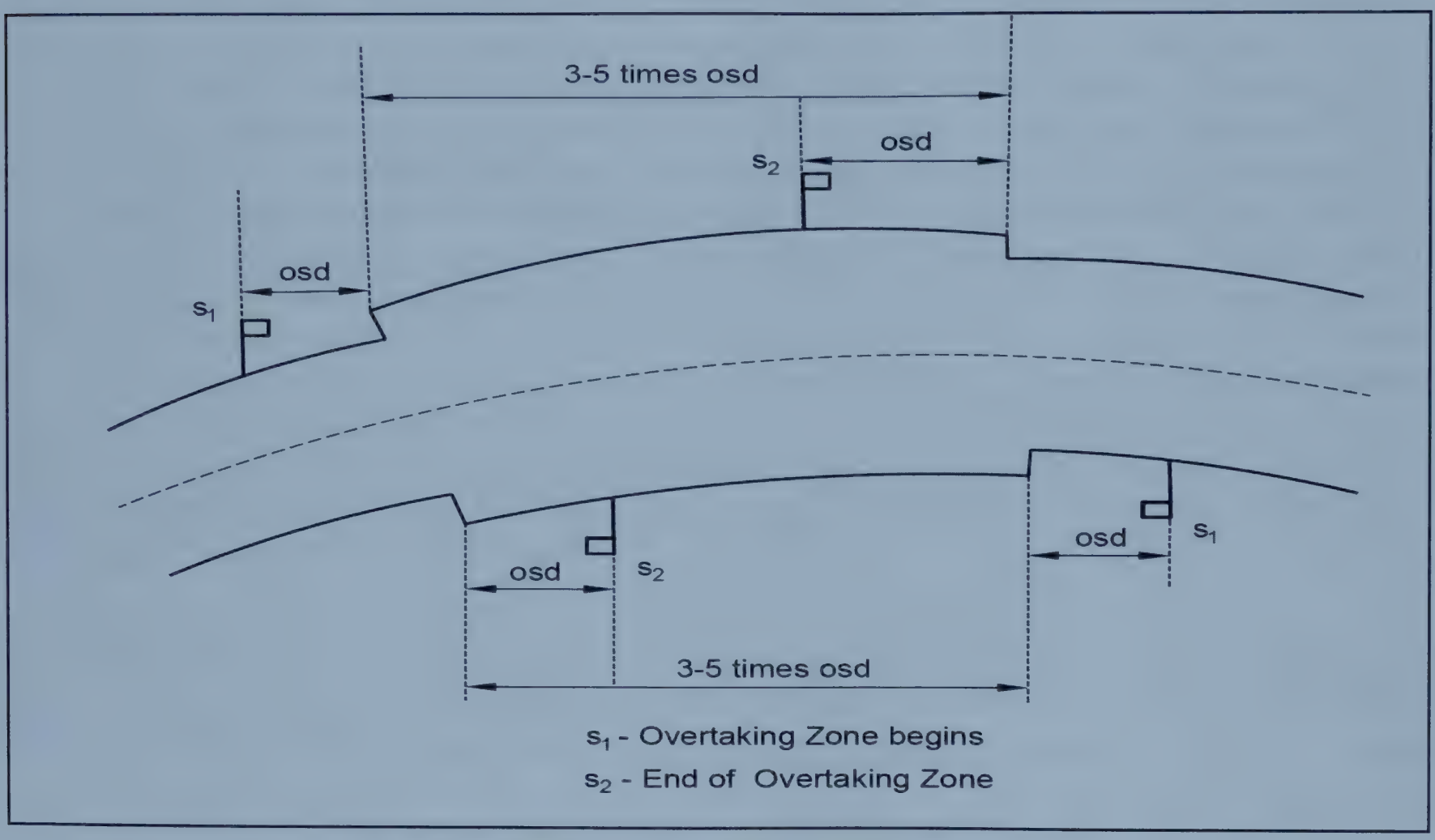


Fig. 5.5 Provision of Overtaking Zones

5.5 Intermediate Sight Distance

5.5.1 *Design Values*

Sections of roads where the customary overtaking sight distance cannot be provided should be designed, as far as possible, for intermediate sight distance which is defined as twice the safe stopping sight distance. This is also known as desirable minimum sight distance. It is based on the concept that intermediate sight distance improves visibility appreciably and affords a reasonable chance to drivers to overtake with caution. Recommended intermediate sight distance/desirable minimum sight distance values for different speeds are given in **Table 5.2**.

5.6 Application of Sight Distance Standards

5.6.1 *Single/Intermediate/Two-Lane Roads*

On single carriageways with two-way traffic (i.e., undivided roads of single-lane or intermediate-lane or two-lane width), the attempt should normally be to provide overtaking sight distance in as much length of the road as possible as per **Table 5.3**. No hard and fast rule can be laid down for the application of overtaking sight distance since this will depend on site conditions, economics etc. Conditions ideal for this application will be:

- Straight sections of road with isolated over-bridges or summit vertical curves where the provision of overtaking sight distance would result in unobstructed sight distance over a long length of the road; and
- Relatively easy sections of terrain adjacent to long reaches with no opportunity for overtaking at all, e.g., at the ends of an excessively winding road in hilly/rolling terrain

In sections, where application of overtaking sight distance is considered impractical for reasons of economics or otherwise, as in an undulating terrain, as far as feasible; sight distance equivalent to at least twice the safe stopping sight distance (i.e., desirable minimum sight distance) should be available throughout as given in **Table 5.2**.

Where visibility hindrance corresponds to these conditions, drivers should be cautioned about the limited sight distance for overtaking through appropriate speed limit signs. The posted speed should be such that the overtaking maneuver can be completed with adequate safety, vide **Table 5.3**.

At summit curves and horizontal curves not satisfying requirements of even the intermediate sight distance, it will be necessary to provide restrictive pavement markings in accordance with IRC:35 "Code of Practice for Road Markings". Where the road section involved is long, "No Overtaking" signs should be installed at the beginning and at intervals.

In no case, however, the visibility should be less than the safe stopping sight distance, which is the basic minimum requirement for any road.

5.6.2 *Divided Highways*

On divided highways with 4 or more lanes, it is not necessary to provide overtaking sight distance as required for single carriageways with-two-way traffic. The overtaking operation can be completed within the multiple lanes in the direction of the travel.

For all the new 4/6/8-lane divided carriageways, the desirable minimum sight distance vide **Table 5.2** must be ensured at all points along the highway. The safe stopping sight distance shall be permitted only on existing 4/6/8-lane divided carriageways where further improvement is not possible due to sight conditions. In fact, it is a good practice to design for values that are somewhat more liberal, to make allowance for the time a driver takes to recognize a stopped vehicle or a dangerous object ahead.

5.6.3 *Undivided Four-Lane Highways*

On undivided four-lane highways there are sufficient opportunities for overtaking within one half

of the carriageway, and there should be no need to cross the center line unless the capacity of the road is grossly deficient. Such roads may, therefore, be designed on the lines of divided highways, i.e., vide **5.6.2** above.

5.6.4 *Expressways*

A desirable minimum sight distance of 500 m corresponding to design speed of 120 kmph shall be available throughout.

5.6.5 *Grade-Change Locations*

The design at all grade-change locations such as vehicular underpass, bridge locations, ROB/ RUB, loops & ramps, etc. must be done using intermediate sight distance i.e. desirable minimum sight distance. In case of expressways, the vertical design, especially at grade change locations, such as VUP, ROB, bridge locations will be done using desirable minimum sight distance of 500 m.

5.7 Intersection Sight Distance

5.7.1 *General*

Visibility is an important requirement at intersections. To avoid collisions, it is essential that sufficient sight distance is available along the intersecting roads and their corners, to enable the operators of vehicles simultaneously approaching the intersection to see each other in time.

At-grade intersections can be broadly grouped under two headings:

- 'Uncontrolled intersections' where the intersecting roads are of nearly equal importance and there is no established priority;
- 'Priority intersections', minor-major road intersections, where one road (major road) takes virtual precedence over the other; Traffic on the minor road may be controlled by STOP or GIVE WAY signs/road markings, making it clear that the other road has the priority.

5.7.2 *Sight Triangles*

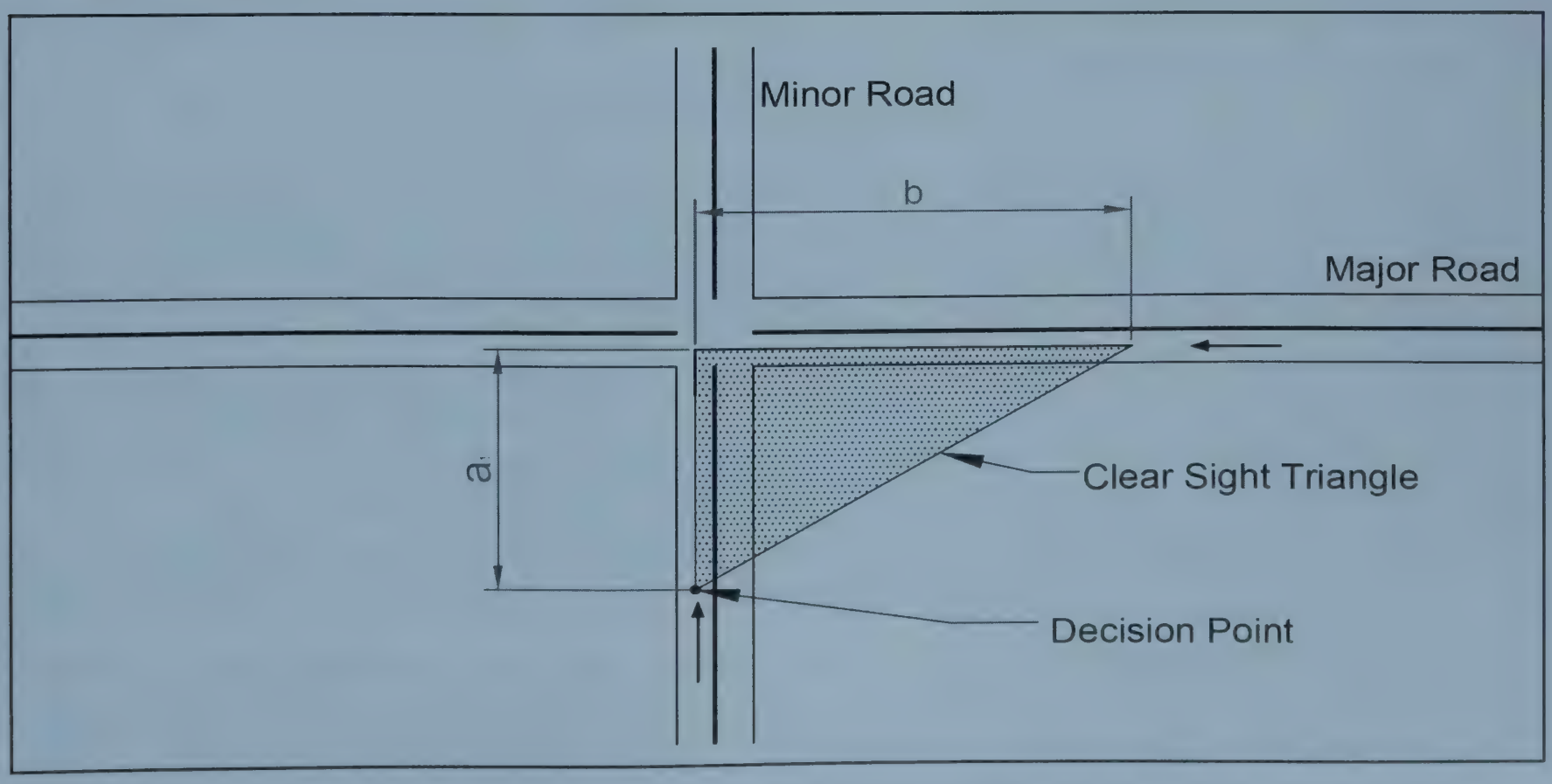
Sight triangles are the specified areas along the intersection approaches (legs) and the included corners within which the line of sight from the vehicles of two adjacent approaches lie. Length of the leg of the sight triangle is the distance of the intersecting point from the decision point. The vertex of the sight triangles is known as the decision point which varies as per the control strategy of the intersection and the design speed of the roads. There are two types of sight triangles based on the traffic control condition at the intersection – approach sight triangle and departure sight triangle. These two sight triangles vary in size (i.e., the length of the leg varies) depending upon the design speed of the intersecting roadways and the traffic control condition at the intersection. The included corners of these sight triangles must be kept free of all obstructions to sight, as illustrated in **Figs. 5.6** and **5.7**. Such obstructions may be buildings, parked vehicles, roadway structures, roadside hardware, hedges, trees, bushes, unmowed vegetation, tall crops, walls, fences, the terrain itself, etc. If visibility conditions up to these standards are ensured, drivers of vehicles will be able to either stop or adjust their speed in anticipation of development of a dangerous situation ahead.

5.7.2.1 Approach Sight Triangles

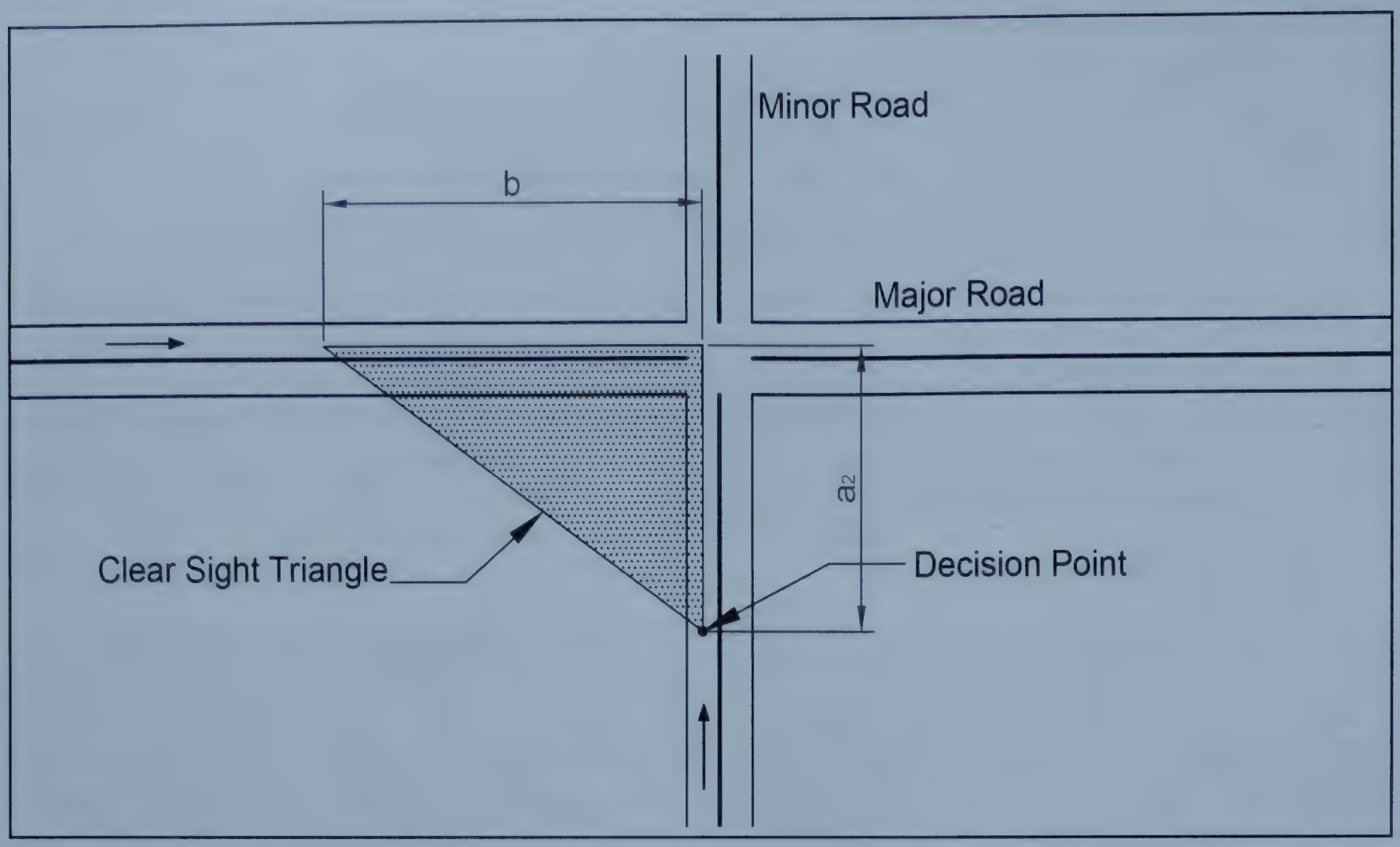
Approach sight triangle represents the triangular area formed by the length of the legs along the two intersecting roadways, which is sufficient for the drivers of vehicles, travelling at design speeds, to see any potentially conflicting vehicle approaching from the other roadway and slow down or stop before colliding at the intersection. This sight triangle is required at two types of intersections – (i) an uncontrolled intersection, where two equally important roadways intersect and (ii) a yield controlled intersection, where a major road intersects with a minor road having a GIVE WAY sign installed along the minor road. The decision point in this triangle represents the position on the roadway where the drivers decide to slow down the vehicle on seeing another vehicle approaching the intersection from the other road and stop, if necessary. From the point of view of any one approach road, there exists two such approach triangles for bi-directional traffic movement, as shown in **Figs. 5.6 (a)** and **5.6 (b)**.

5.7.2.2 Departure Sight Triangles

Departure sight triangle represents the triangular area formed by the length of the legs along the two intersecting roadways, which is sufficient for the driver of a stopped vehicle on the minor road to locate a vehicle travelling at design speed along the major road and decide to enter or cross the major road without colliding within the intersection. This sight triangle is required at major-minor road intersection, where traffic on a minor road is controlled by STOP sign and/or STOP marking. In this case, the decision point is entirely based on the location of the STOP sign installed or the STOP line marking on the minor road. When a sufficient time gap is available within traffic stream of major road, the driver of the vehicle stopped along the minor road decides to accelerate the vehicle and cross the major road. From the point of view of the driver on minor road, there exists two such departure triangles for bi-directional traffic movement as shown in **Figs. 5.7 (a)** and **5.7 (b)**. Additionally, the clear departure triangle allows the drivers of vehicles on the major road to detect any stopped vehicle on the minor road and be prepared to slow down or stop, if necessary.

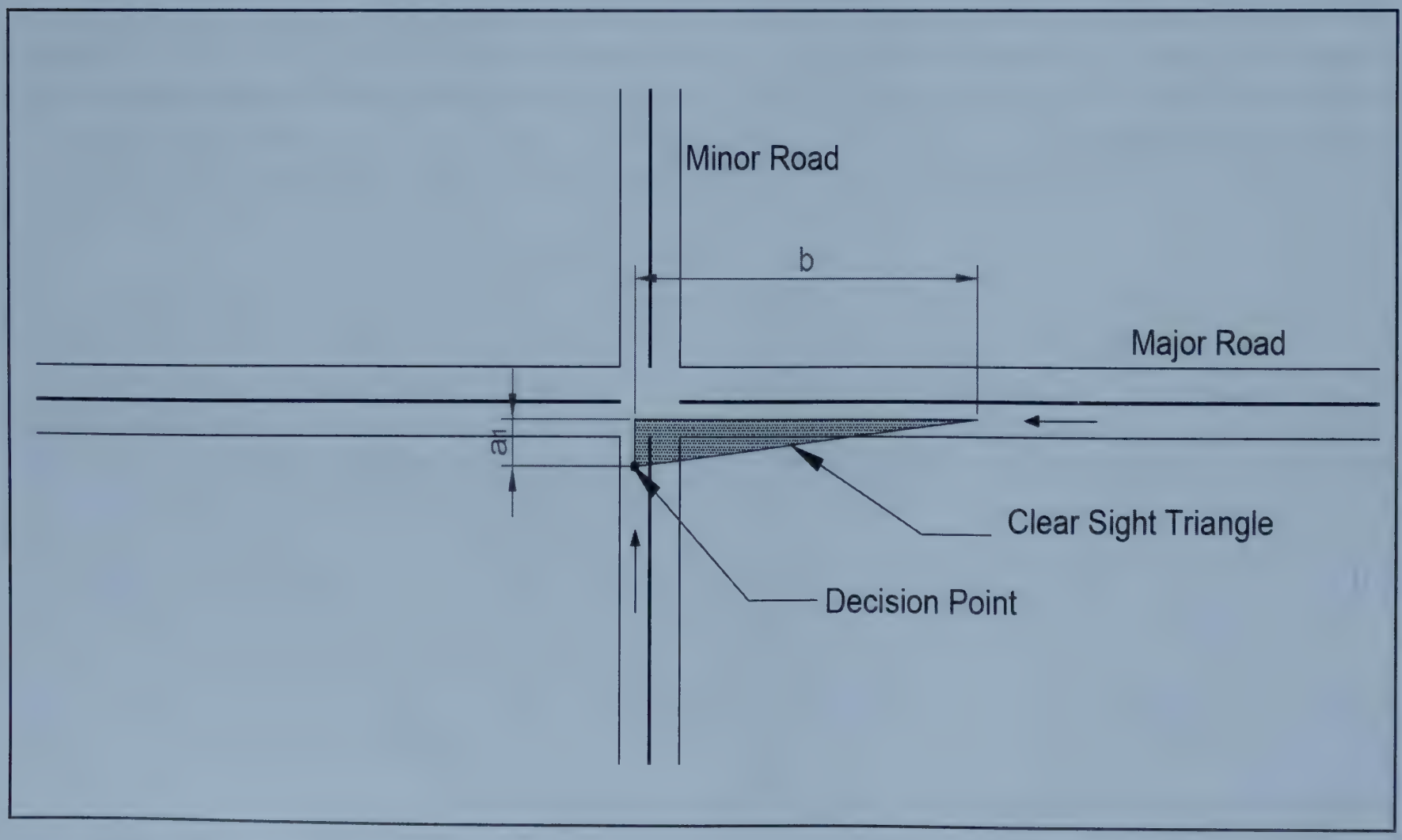


a) Approach Sight Triangle for Major Road corresponding to Minor Road on Left

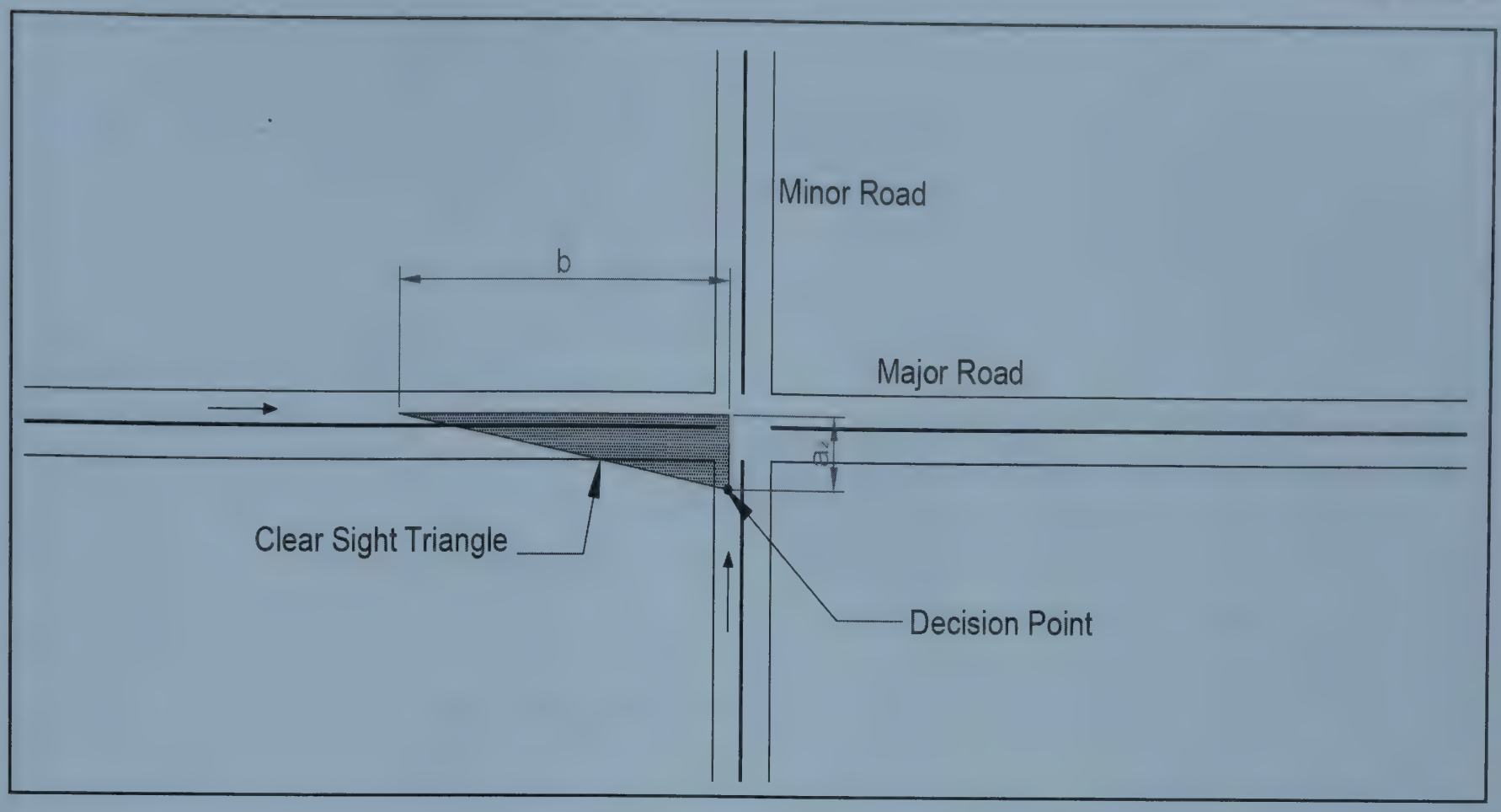


b) Approach Sight Triangle for Major Road corresponding to Minor Road on Right

Fig. 5.6 Approach Sight Triangles at Uncontrolled or Yield Controlled Intersections



a) Departure Sight Triangle for Major Road corresponding to Minor Road on Left



b) Departure Sight Triangle for Major Road corresponding to Minor Road on Right

Fig. 5.7 Departure Sight Triangles at Stop Controlled Intersections

5.7.3 *Uncontrolled Intersections*

5.7.3.1 Design Criteria and Design Values

At these intersections, visibility should be provided on the principle that the drivers of vehicles on either approaches are able to see the intersection and the intersecting roadways in good time to be able to stop the vehicles, if that becomes essential. At such intersections, the area for the clear visibility must be available as per the approach sight triangle criteria. The approach sight triangles in the included corners of uncontrolled intersections, which must be kept free of all obstructions to sight, could be demarcated as illustrated in **Fig. 5.6.**

The detection and recognition of a vehicle approaching the intersection from a substantial distance may require a time of 2.5 sec, within the limits of the driver's peripheral vision. In Indian context, it is observed that the vehicles approaching the intersection typically reduce their speed to 60% of the design speed which gives additional safety at the intersection Therefore, the approach sight triangle formed with the length of legs equal to the desirable minimum sight distance for each highway corresponding to the design speed would suffice, as given in **Table 5.2**. For example, if the two roadways intersect at an uncontrolled junction having design speeds of 80 kmph and 60 kmph, then the length of the legs along the two approaches must be 260 m and 90 m respectively and thereby form the approach sight triangle as shown in **Fig. 5.8** within which there should not be any obstructions whatsoever.

If it is not possible to accommodate the size of the approach sight triangle based on the desirable minimum sight distance criterion, due to space constraints and other unavoidable circumstances, then in such situations, the length of the legs of the approach sight triangle must not be less than the safe stopping sight distance.

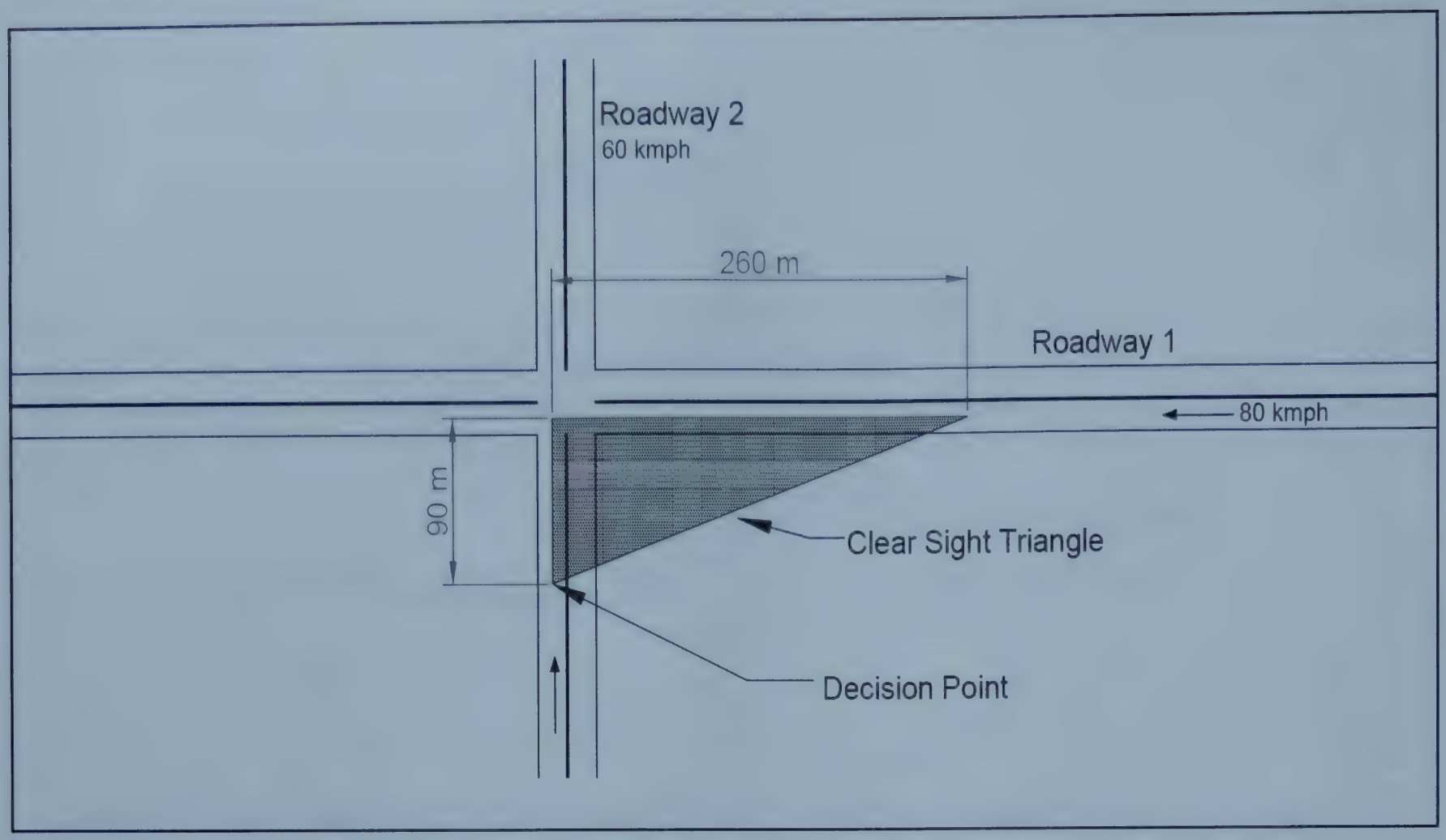


Fig. 5.8 Approach Sight Triangle for Roadways with Design Speeds of 80 kmph and 60 kmph

The approach sight triangle for major district roads, other district roads and village roads may be formed with the length of legs equal to the safe stopping sight distance for each road corresponding to the design speed, as given in **Table 5.2**.

5.7.3.2 Constrained Visibility

Occasionally, the legs of the sight triangle may be even less than the safe stopping sight distance due to presence of an obstruction which cannot be removed except at prohibitive cost. Under such circumstances, there can be two solutions: (i) permit vehicles on one of the roads to travel at the design speed and estimate the corresponding reduced speed for the other road which must be posted speed limit; (ii) the approach speed for both the roads could be restricted in accordance with the sight triangle available by installing suitable speed limit signs. The adoption of the solution will entirely depend on the site condition and should be addressed judiciously.

5.7.4 *Priority Intersections*

On priority intersections, the visibility provided should be such that drivers approaching from the minor road are able to see the vehicles on the major road within adequate time and to judge whether the required time gap available in the major road traffic stream is safe for crossing. Otherwise, the vehicle could be brought to a halt, if necessary. Visibility distance along the major road for priority intersections is governed by the type of control exercised on minor road. The control maybe (i) Stop Controlled,or (ii) Yield Controlled. In case of 2/4 lane highways, the control on the minor road should be 'Stop Control' unless specified otherwise. Since, the access to 6/8 lane highways is access controlled, there should not be any at-grade stop-controlled intersections for such highways.

5.7.4.1 Stop Control on Minor Road

At these intersections, visibility should be provided on the principle that the drivers of stopped vehicles on the minor road are able to see the vehicles approaching the intersection from the major road and assess the time gap within which the driver may accelerate and cross the major road safely. At such intersection, the area for clear visibility must be available as per the departure sight triangle criterion. The departure sight triangles in the included corners of the stop-controlled intersections, which must be kept free of all obstructions to sight, could be demarcated as illustrated in **Fig. 5.7**.

The decision point for stop controlled minor road is dependent upon the position of the installed STOP sign or the STOP line marking. Position of the STOP line marking varies from road to road and may be considered as per IRC:35 and IRC:67. Therefore, the length of the leg of departure sight triangle along the minor road varies as per the placement of the STOP line marking.

In stop controlled intersection, vehicles from the minor road accelerate from a stationary position and perform either of the three movements–Left turn to merge with major road; Right turn to merge with the major road traffic with the lanes beyond the median; Cross the major road. The time gap required for each of these three individual movements maybe different, but the highest time gap out of the three movements will be considered as sufficient for all the movements. The time gap is also dependent on the type of the vehicle negotiating the intersection from the minor road.

In the Indian context, on a two-lane highway, a time gap of 8 seconds is considered sufficient for a standard passenger car on the minor road to cross or turn right or turn left safely without hampering the flow of the major road. The concept for arriving at 8 seconds is provided as below. **Table 5.4** shows the time gaps for different vehicle types.

The distance required to cross will depend on the width of roadway, length of the vehicle and distance from the edge of carriageway to the stop line as given below:

- The distance from the edge of carriageway to the stop line may be taken as 3 m (as per IRC:67 for lightly trafficked road). However, this distance may vary as discussed earlier
- The length of the vehicle (i.e., for standard car) is taken as ≈ 4.7 m
- Width of the roadway is equal to total lane widths, median width and shoulder widths. For a two-lane highway, this may be taken as 7 m lane widths plus 3 m (1.5 m + 1.5 m) paved shoulder width.

Therefore, the total distance required to cross along a two-lane highway will be 17.7 m. As the sufficient gap becomes available, vehicle will accelerate and achieve a constant speed and cross the road. It is assumed that this constant speed is equivalent to minimum design speed of 20 kmph. Based on the various studies related to acceleration and deceleration profiles of vehicles on rural highways conducted in India, it is observed that, a standard car accelerates at an average rate of 1.5 m/s^2 between 0 to 20 kmph.

Therefore, total time required to cross the road is equivalent to sum of perception-reaction time plus time taken to cross the major road. It is as given below:

- Perception-reaction time may be taken as 2.5 s
- The time taken to achieve the design speed of 20 kmph at an acceleration of 1.5 m/s^2 is equal to 3.71 s. During 3.71 s, the distance travelled by vehicle will be 10.3 m.

- The remaining distance (17.7-10.3= 7.4 m) is travelled at 20 kmph and will take 1.33 s.

Therefore, the total time required for crossing totals out to 7.54 s ≈ rounded off to 8 seconds. Visibility distance to ensure a clear sight triangle should be equivalent to distance travelled by the vehicle on major road during these 8 seconds. Further, it is suggested that the time taken by vehicle on minor road for left turn or right turn should also be taken as equal to the total time required for crossing.

Table 5.4 Time Gap for different Types of Design Vehicles

Design Vehicle	Time Gap at Design Speed of Major Road (s)
Passenger Car	8.0
Heavy Vehicles (Trucks)	10.0

For multi-lane highways, the time gap must be increased by 0.5 seconds for passenger cars and 0.7 seconds for trucks for each additional lane beyond two lanes. If median is present, the width of the median must be converted to equivalent lane width and the additional time gap of 0.5 sec per lane for passenger car and 0.7 sec per lane for trucks must be increased.

The lengths of legs of the departure sight triangle along the major road of a stop-controlled intersection are suggested in **Table 5.5**. The values are based on the time gap required for passenger car which forms the departure sight triangle as shown in **Fig. 5.7** within which there should not be any obstructions whatsoever. Ideally, the length of the leg should be the maximum of (i) desirable minimum sight distance as per **Table 5.2** and (ii) length of the leg along the major road as per **Table 5.5**. However, in no case it should be less than the design values provided in **Table 5.5**. The clear departure sight triangle will allow the drivers of vehicles on the major road to detect any stopped vehicle on the minor road and be prepared to slow down or stop, if necessary. This sight triangle is sufficient for vehicles on major road intersecting with vehicles on near side minor road as well as far side minor road.

Table 5.5 Length of Leg along the Major Road required for Safe Crossing of Passenger Car from the Minor Road

Design Speed of Major Road (kmph)	'b'=Length of the Leg along Major Road (Meters)			
	2-Lane Road		4-Lane Road	
	Calculated Value (m)	Design Value (m)	Calculated Value (m)	Design Value (m)
20	44	45	56	55
25	56	55	69	70
30	67	70	83	85
40	89	90	111	110
50	111	110	139	140
60	133	135	167	170
65	144	145	181	180
80	178	180	222	220
100	222	220	278	280

Note: *Since, the access to 6/8 lane highways is access controlled, there should not be any at-grade stop-controlled intersections for such highways.*

The departure sight triangle for major district roads, other district roads and village roads intersecting with any minor road may be formed with the length of legs corresponding to the design speeds as given in **Table 5.5**.

5.7.4.2 Yield Control on Minor Road

At yield controlled intersections, it is not mandatory for the vehicles travelling along minor road to stop completely and it is not mandatory for the vehicles travelling along the major road to reduce their speed. The vehicle on the minor road needs to assess the situation by slowing down and is permitted to enter or cross the major road without stopping if there are no potentially conflicting vehicle approaching the intersection from the major road. At these intersections, visibility should be provided on the principle that the drivers of vehicles on minor road travelling at a reduced speed are able to see the intersection and the intersecting major road/highway in good time to be able to halt and give way to the major road traffic. At such intersections, the area for clear visibility must be available as per the approach sight triangle criterion. The approach sight triangles in the included corners of uncontrolled intersections, which must be kept free of all obstructions to sight, could be demarcated as illustrated in **Fig. 5.6**.

In yield controlled intersections, vehicles from the minor road perform either of the three movements – Left turn to merge with major road; Right turn to merge with the major road traffic on the lanes beyond the median; Cross the major road. In order to safely complete either of these movements, sufficient time gap should be available in the major road traffic to allow the vehicle of the minor road to go through two important steps – (i) to travel from the decision point to the intersection, while decelerating at the rate of 1.5 m/s^2 and reaching 60% of the minor road design speed, and (ii) to cross and clear the intersection maintaining the same speed.

The distance between decision point to the intersection (let's say 'SD') is taken as equal to the safe stopping sight distance corresponding to design speed on minor road. During this approach, the driver on the minor road perceives the traffic conditions at the intersection, evaluates the gaps in the vehicle stream on major road, takes a decision about actual crossing, and if the sufficient gap is available, vehicle crosses the major road at a speed corresponding to 60 percent of design speed.

The total time (t) required for these operations includes the time taken for vehicle to travel along the approach length (t_1) plus time taken to cross the major road (t_2).

- The time, t_1, can be estimated on the basis of initial speed equivalent to design speed and initial rate of deceleration of 1.5 m/s^2 based on the equations of motion. The time, t_1, itself has two components: t_{1a} and t_{1b}
    - As the vehicle decelerates at 1.5 m/s^2 from the decision point, the vehicle will reach the 60% design speed before reaching the intersection and the vehicle will travel a distance 'S_1' during this time (t_{1a})
    - The remaining distance 'SD –S_1' will be travelled at 60% design speed in time (t_{1b})
- The time, t_2, can be estimated based on 60% of design speed and distance of crossing (equivalent to width of major road roadway + length of vehicle).

The available visibility distance along the major road (length of leg along the major road = 'b') should be equal to distance travelled by vehicle on major road within total time (t) at the design speed along the major road. It must be ensured that the total time (t) must not be less than the crossing time requirements corresponding to stop-controlled intersection.

The design values corresponding to minimum visibility distance along major roads (i.e., distance 'b') should be given as per **Tables 5.6** and **5.7**. Ideally, the length of the leg should be the maximum of (i) desirable minimum sight distance as per **Table 5.2**, and (ii) length of the leg along major road as per **Tables 5.6** and **5.7**. However, in no case it should be less than the design values provided in **Tables 5.6** and **5.7**. On this basis, the sight triangle at yield-controlled intersections should be formed as illustrated in **Fig. 5.6** within which there should not be any obstructions whatsoever. The clear approach sight triangle will allow the drivers of vehicles on the major road to detect any vehicle from the minor road approaching the intersection and be prepared to slow down or stop, if necessary. This sight triangle is sufficient for vehicles on major road intersecting with vehicles on near side minor road as well as far side minor road.

Table 5.6 Length of the Leg along Major Roads at Yield-Controlled Intersections (2-Lane Major Road)

Minor Road			Total travel time for crossing (s)	Design Value (s)	Major Road Design Speed (kmph)					
Design Speed (kmph)	Length of the leg (m)	Time along leg (s)			40	50	60	65	80	100
					Length of the Leg along Major Road (m)					
20	20	1.5	10.8	10.8	120	150	180	195	240	300
25	25	1.9	9.6	9.6	105	135	160	175	215	270
30	30	2.2	8.8	8.8	100	120	145	160	195	245
40	45	3.0	8.4	8.4	95	115	140	150	185	235
50	60	3.7	8.1	8.1	-	110	135	145	180	225
60	80	4.4	8.3	8.3	-	-	140	150	185	230

Table 5.7 Length of the Leg along Major Roads at Yield-Controlled Intersections (4-Lane Major Road)

Minor Road			Total travel time for crossing (s)	Design Value (s)	Major Road Design Speed (kmph)					
Design Speed (kmph)	Length of the leg (m)	Time along leg (s)			40	50	60	65	80	100
					Length of the Leg along Major Road (m)					
20	20	1.5	11.3	11.3	125	155	190	205	250	315
25	25	1.9	9.7	10.0	110	140	165	180	220	280
30	30	2.2	8.8	10.0	110	140	165	180	220	280
40	45	3.0	7.9	10.0	110	140	165	180	220	280
50	60	3.7	7.6	10.0	-	140	165	180	220	280
60	80	4.4	7.7	10.0	-	-	165	180	220	280

Note: *Since, the access to 6/8 lane highways is access controlled, there should not be any at-grade yield-controlled intersections for such highways.*

The approach sight triangle at yield-controlled intersections for major district roads, other district roads and village roads intersecting with any minor road may be formed with the length of legs corresponding to the design speeds as given in **Tables 5.6** and **5.7**.

5.8 Headlight Sight Distance at Valley Curves

5.8.1 *General*

During day time, visibility is not a major issue on valley curves. However, for travel at night the design must ensure that the roadway ahead is illuminated by vehicle headlights to a sufficient length, enabling the vehicle to brake to a stop if necessary. This distance, called the headlight sight distance, should at least equal the safe stopping sight distance given in **Table 5.2**.

From safety considerations, it is essential that valley curves should be designed to provide for this visibility. **Fig. 5.9** shows a typical case of headlight sight distance on a sag vertical curve where sight distance is greater than the length of valley curve.

5.8.2 *Criteria for Measurement*

For designing valley curves, criteria to be followed to ensure the headlight sight distance are mentioned below:

- height of headlight above the road surface is 0.75 m;
- the useful beam of headlight is one degree upwards from the grade of the road; and
- the height of object is nil

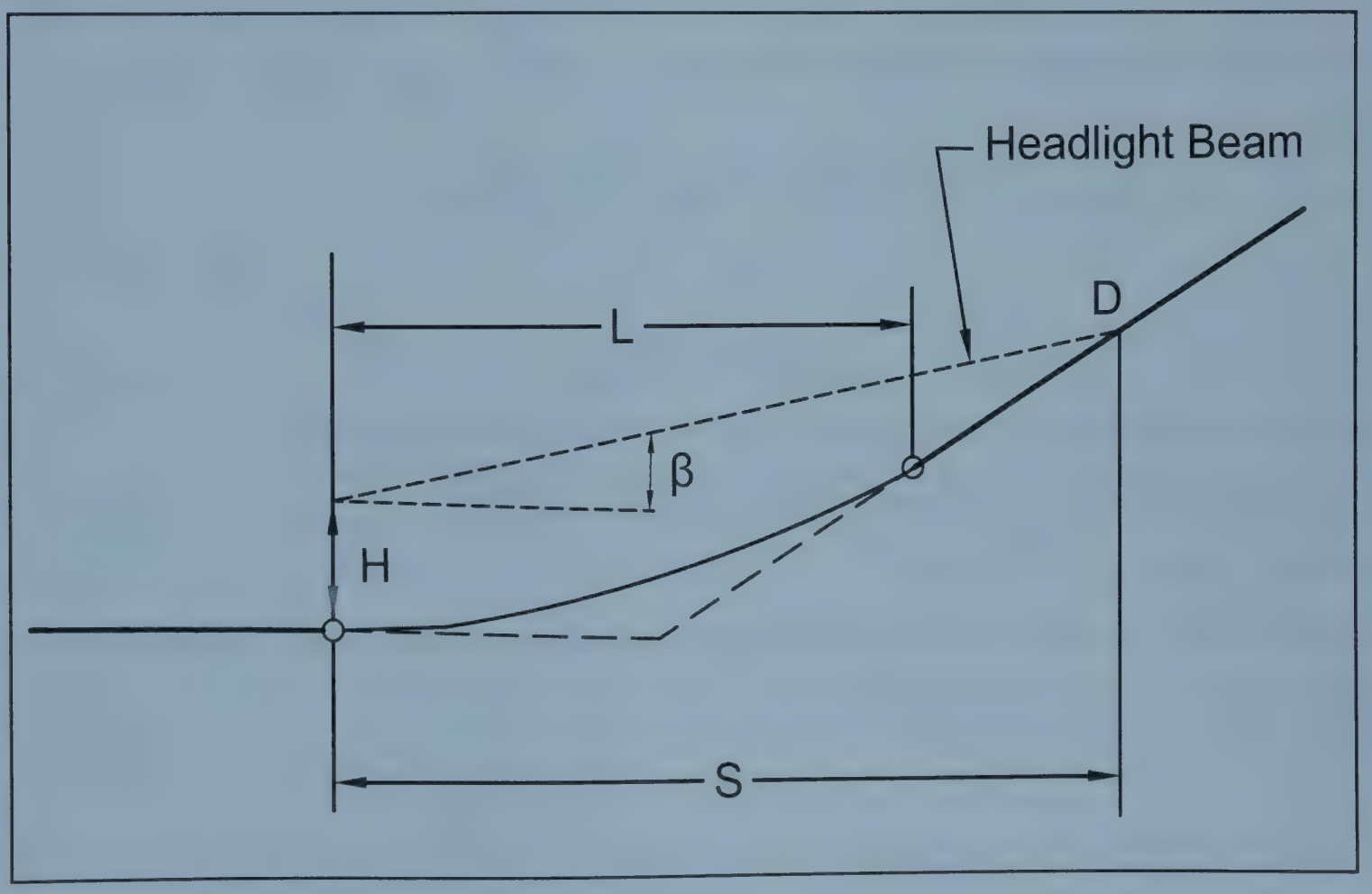


Fig. 5.9 Headlight Sight Distance in Sag Vertical Curve (S>L)

5.9 Measuring and Recording Sight Distance

Provision of the required sight distance should receive care right from the early stages when the alignment of a highway is still flexible and subject to adjustments. Quick appraisals are best done by graphical means. By determining the available sight distance graphically from plans and

profile drawings, and recording it at convenient intervals, deficiencies in visibility can be detected in time so that necessary modifications could be made before detailed design.

Sight distances on the planned roadway may be observed by direct scaling and can easily be found out where plans and profiles are drawn. Sight distance can be easily determined where plans and profiles are drawn using Computer-Aided Design and Drafting (CADD) systems. Horizontal sight distances can be directly scaled from plans on which obstructions to visibility such as buildings, plantation, hill slopes etc., have been marked. The measurement is done with the help of a straight edge.

Measurement of vertical sight distance may be done graphically from plotted profiles of the highway using CAD-based or similar design software. The visibility/sight distance shall be checked using design software after completion of entire design including super-elevation, placement of barriers, etc. The design shall be modelled with all the proposed design elements including the structure parapets. The visibility analysis shall be carried out (using design software only) and the resultant visibility should be checked for any obstruction or the design may be tweaked accordingly.

5.10 Illustrative Example

Example on Stopping Sight Distance

Find the stopping sight distance for a design speed of 70 km/h, assuming suitable data. What are the sight distance requirements at a gradient of 1 in 40?

Solution:

The stopping sight distance can be obtained using the following equations:

$$d_{SSD} = d_1 + d_2$$

$$d_{SSD} = 0.278\ Vt + \frac{V^2}{254f}$$

d_1 = The distance travelled between perception and brake reaction time

d_2 = The braking distance once the brakes have been applied

Given Design speed (V) = 70 kmph

Assuming reaction time *(t)* = 2.5 seconds, and coefficient of friction *(f)*= 0.36 from **Table 5.2**

Therefore,

$$d_{SSD} = 0.278*70*2.5 + \frac{70^2}{254*0.36}$$

$$d_{SSD} = 48.65 + 53.59$$

The distance travelled between perception and brake reaction time is 48.65 m

The braking distance once the brakes has been applied is 53.59 m

Therefore, the total stopping sight distance on a level gradient is

$$d_{SSD} = 102.24\ m\ (say\ 105\ m)$$

In case of an upward gradient, G will be positive

$$d_2 = \frac{V^2}{254\,(f \pm 0.01G)}$$

$$\text{Given, } G = \frac{1}{40} = 2.5\%$$

Therefore, the braking distance for upward gradient is

$$d_2 = \frac{70^2}{254\,(0.36 + 0.01 * 2.5)}$$

$$d_2 = 50.11\ m$$

Therefore, the total stopping sight distance on an upward gradient is 48.65 + 50.11 = 98.76 m (Say 100 m)

In case of an downward gradient, G will be negative

$$d_2 = \frac{V^2}{254\,(f \pm 0.01G)}$$

Therefore, the braking distance for downward gradient is

$$d_2 = \frac{70^2}{254\,(0.36 - 0.01 * 2.5)}$$

$$d_2 = 57.59\ m$$

Therefore, the total stopping sight distance on an upward gradient is 48.65 + 57.59 = 106.24 m (Say 110 m)

6. HORIZONTAL ALIGNMENT

6.1 Introduction

Uniformity of design standards is one of the essential requirements of a road alignment. In a road alignment, changes in direction are often necessary due to restrictions imposed by topography, environmental and ecological characteristics of areas, presence of monuments, places of worship, sites of structures and other considerations such as availability of land. For convenient, safe and pleasant travel on the road, changes in directions have to be accomplished by introducing suitable horizontal curves in the alignment.

The general principle of design to be followed while using horizontal curves is to coordinate both the horizontal and the vertical alignments. Also, the horizontal curves should be fluent and blend well with the surrounding topography. Some of the important considerations to be followed while introducing horizontal curves are summarized below:

a. Sharp horizontal curves should be avoided at or near the apex of summit/sag vertical curves from safety considerations.

b. Vertical curvature superimposed upon horizontal curvature gives a pleasing effect. As such the vertical and horizontal curves should coincide as far as possible and their length should be more or less equal. If this is difficult for any reason, the horizontal curve should be somewhat longer than the vertical curve and vice-versa.

c. The degree of curvature should be in proper balance with the gradients. Straight alignment

or flat horizontal curves on steep or long grades, or excessive curvature on a road with flat grades, do not constitute balanced designs and should be avoided.

d. Long tangent sections exceeding 3 km in length should be avoided as far as possible. A curvilinear alignment with long spirals is better from the point of safety and aesthetics. This is the first and the most important point in fixing the alignment.

e. Sudden change in alignment, for example sharp curves should not be introduced at the end of long tangents since these can be extremely hazardous.

f. Short curves give appearance of kinks, particularly for small deflection angles, and should be avoided. The curves should be sufficiently long and have suitable transitions to provide pleasing appearance. Curve length should be at least 150 metres for a deflection angle of 5 degrees, and this should be increased by 30 metres for each one-degree decrease in the deflection angle.

g. Reverse curves may be needed in difficult terrain. It should be ensured that there is sufficient length between the two curves for introduction of requisite transition curves and required super-elevation.

h. Curves in the same direction separated by short tangents, known as broken-back curves, should be avoided as far as possible in the interest of aesthetics and safety and replaced by a single curve. If this is not feasible, a tangent length corresponding to 10 seconds travel time must at least be ensured between the two curves.

i. Compound curves may be used in difficult topography but only when it is impossible to fit in a single circular curve. To ensure safe and smooth transition from one curve to the other, the radius of the flatter curve should not be disproportional to the radius of the sharper curve. The radius of the flatter curve shall be at the maximum of 1.5 times the radius of the sharper curve.

j. To avoid distortions in appearance, the horizontal alignment should be coordinated carefully with the longitudinal profile, keeping in mind that the road is a three-dimensional entity and does not consist simply of a plan and L-section. Requirements in this regard are discussed in **Section 8**.

k. The siting of the bridges and the location of the approaches should be properly coordinated keeping in view the overall technical feasibility, economy, fluency of alignment and aesthetics.

The following criteria may be followed in general:

i. For major bridges above 300 m span, proper siting of the bridge should be the principal consideration and the approach alignment matched with the same.

ii. For small bridges less than 60 m span, fluency of the alignment should govern the choice of the bridge location; and

iii. For spans between 60 m and 300 m, the designer should use his discretion keeping in view the importance of the road, overall economic considerations and aesthetics

6.2 Definitions and Notations

Fig. 6.1 shows all the elements of a combined circular and transition curve mentioned below.

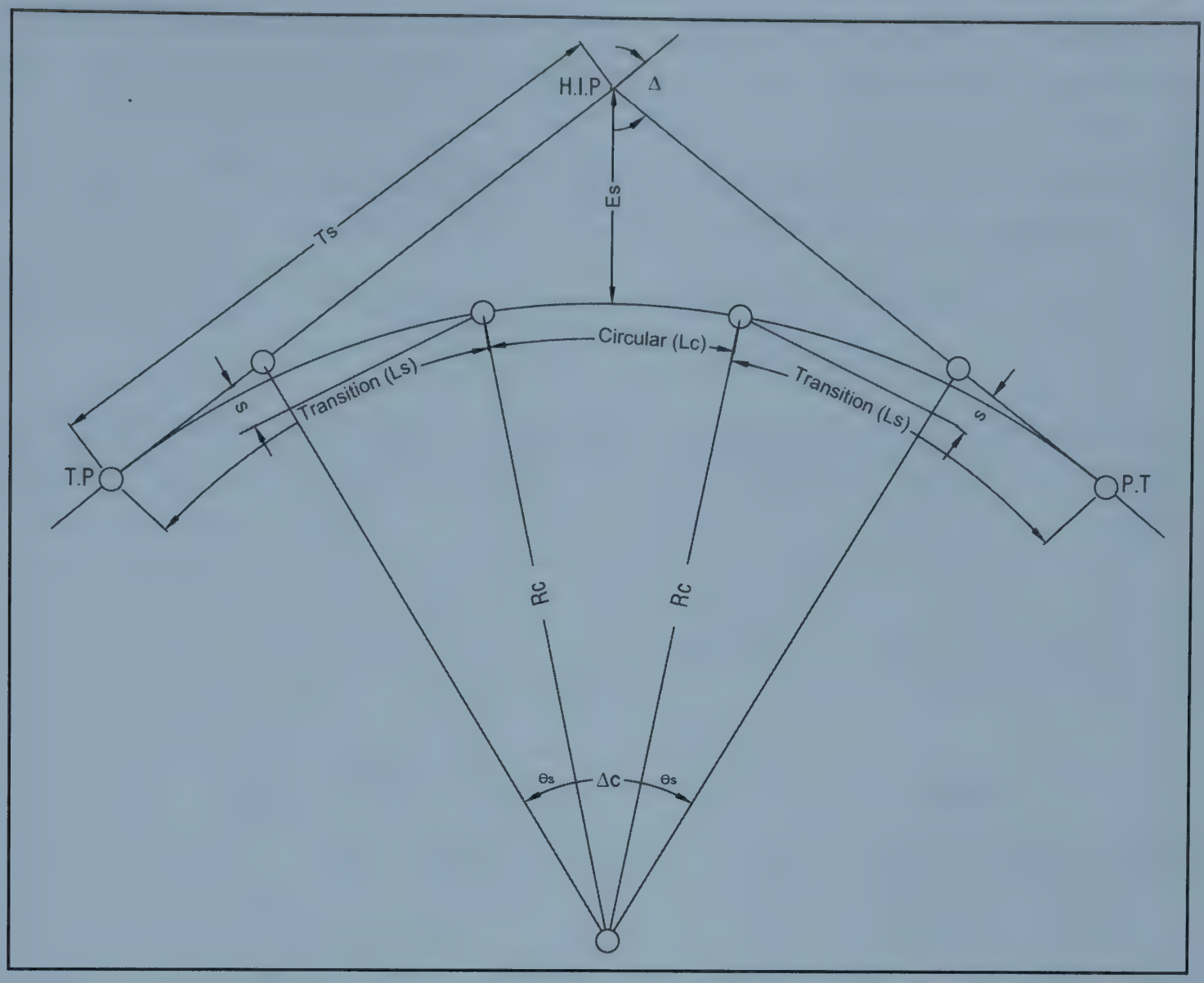


Fig. 6.1 Elements of a Combined Circular and Transition Curve

Tangent Point (T.P. or P.T.) - The point where the straight alignment ceases and curvature begins. Also, the point of reversal of curvature

Horizontal Intersection Point (H.I.P.) - The point of intersection of the two straights tangents connected by curve.

Total Deviation Angle (Δ) - The external angle at the intersection between the tangents at the ends of a curve. This measures the total change of direction in the alignment of the route. This is also the central angle of the complete curve.

Apex Distance (E_s) - The shortest distance from the apex to the curve.

Length of Transition (L_s) - The full length of the transition curve connecting a straight length of road with a curve which may be circular or transitional.

Tangent Distance (T_s) - The length of the straight between the apex (P.I.) and the tangent point of the curve (P.T. or T.P.)

Shift (s) - The displacement of a circular curve from the straight to provide room to introduce a transition curve between it and the straight.

Deviation and Central Angle of Circular Arc (Δ_c) – Deviation angle or central angle of the circular part of a curve with a transition at either end.

Deviation Angle of Transition Curve (θ_s) – The tangent deflection angle for the end of the transition curve, i.e. the external angle between the straight and the tangent to the curve at the end of the transition.

Radius of Circular Curve (R_c) –The radius of a transitional curve at the point where maximum designed super-elevation is reached.

6.3 Design of Circular Curve Radius

6.3.1 *Dynamics of Motion of a Vehicle on a Curve*

When a vehicle travels around a curve of constant radius at constant speed, it exerts radially an outward force known as the 'Centrifugal force'. This centrifugal force (P) can be represented by

$$P = \frac{Wv^2}{gR} \quad \ldots (6.1)$$

$$P = \frac{WV^2}{127R} \quad \ldots (6.2)$$

$$\frac{P}{W} = \frac{v^2}{gR} \quad \ldots (6.3)$$

Where,

W = Weight of the Vehicle

v = Speed of Vehicle, m/sec

g = Acceleration due to Gravity = 9.81 m/sec^2

It acts horizontally at the center of gravity of the vehicle and its load. Centrifugal ratio is defined as the ratio of the centrifugal force to the weight of the vehicle. On the horizontal curve, the centrifugal force is balanced by the combined effects of super-elevation and side friction as shown in **Fig. 6.2**. Mathematically, it can be shown that the centrifugal ratio is equal to sum of super elevation and friction and hence the basic equation for this condition of equilibrium is

$$e + f = \frac{V^2}{127R} \quad \ldots (6.4)$$

where,

V = Design Speed in kmph

e = Super-Elevation in meter per meter

f = Coefficient of Side Friction between Vehicle Tyres and Pavement

R = Radius in meters

6.3.2 *Friction between the Tyre and Road Surface*

The value of the coefficient of lateral friction is a function of speed of the vehicle, the type and condition of road surface, the condition of tyres, the weather conditions at the time of contact between tyre and the road, the temperature of the road, etc. The coefficient of lateral friction is as low as 0.2 when the pavement is covered with mud. The safe value of friction for design of curves is taken as 0.15 allowing a factor of safety of $1\frac{1}{3}$.

6.3.3 *Minimum Curve Radius*

The minimum curve radii are governed by two factors:

- The design speed, allowable super-elevation and friction
- The minimum turning radii of design vehicle

The basic equation for this condition of equilibrium is

$$e + f = \frac{V^2}{127R}$$

- ✓ For expressways and 2/4/6/8 lane highways, maximum allowable super-elevation should be limited to 7 percent, if radius of curve is less than desirable minimum radius. It shall be limited to 5 percent, if radius is more than desirable minimum radius and also at section where expressway/highway passes through an urban section or falls on a major junction.
- ✓ For expressways and 2/4/6/8 lane highways, absolute minimum and desirable minimum radii corresponding to the minimum design speed and ruling design speed respectively are given in below **Table 6.1**.

Table 6.1 Radii of Horizontal Curves for EW/NH/SH

Nature of Terrain	Desirable Minimum Radius	Absolute Minimum Radius
Plain & Rolling (Expressways)	1000 m	650 m
Plain & Rolling (NH/SH)	400 m	250 m
Mountainous & Steep (NH/SH)	150 m	75 m

- ✓ For calculating the minimum radii values given in **Table 6.2**, super elevation values are based on **Table 6.3**, friction factor is taken as 0.15 and, ruling minimum and absolute minimum radii corresponding to the ruling design speed and minimum design speed respectively. Minimum radii of horizontal curves of MDR/ODR/VR for different terrain conditions are given in **Table 6.2**.

Table 6.2 Minimum Radii of Horizontal Curves for MDR/ODR/VR (m)

Classification of Road	Plain Terrain		Rolling Terrain		Mountainous Terrain				Steep Terrain			
					Area not affected by snow		Snow bound areas		Area not affected by snow		Snow bound areas	
	Ruling Minimum	Absolute Minimum	Ruling Minimum	Absolute Minimum	Ruling Minimum	Absolute Minimum	Ruling Minimum	Absolute Minimum	Ruling Minimum	Absolute Minimum	Ruling Minimum	Absolute Minimum
MDR	230	155	155	90	50	30	60	33	30	14	33	15
ODR/VR	90	60	90	60	30	13	33	15	30	13	33	15

6.4 Super-Elevation

6.4.1 *Necessity of Super-Elevation*

In order to counteract the effect of centrifugal force and to reduce the tendency of the vehicle to overturn or skid, the outer edge of pavement or outer edge of paved shoulders (roads with paved shoulders) is raised with respect to inner edge, thus providing a transverse slope throughout the length of horizontal curve. This transverse inclination to the pavement surface is known as super-elevation or cant or banking. The super-elevation 'e' is expressed as the ratio of the height of outer edge with respect to the horizontal width.

6.4.2 *Design value of Super-Elevation*

Super-elevation required on horizontal curves should be calculated from the following formula. This assumes that centrifugal force corresponding to three fourth the design speed (by neglecting the lateral friction developed) is balanced by super-elevation.

$$e = \frac{V^2}{225R} \qquad \ldots (6.5)$$

Where, e = Super-Elevation in meter per meter

V = Design Speed in km/hr

R = Radius in meter

Super-elevation obtained using the above expression shall be not less than the minimum specified camber and the maximum allowable super-elevation is discussed in **Section 6.4.3**. **Fig. 6.2** indicates a typical super-elevated pavement section.

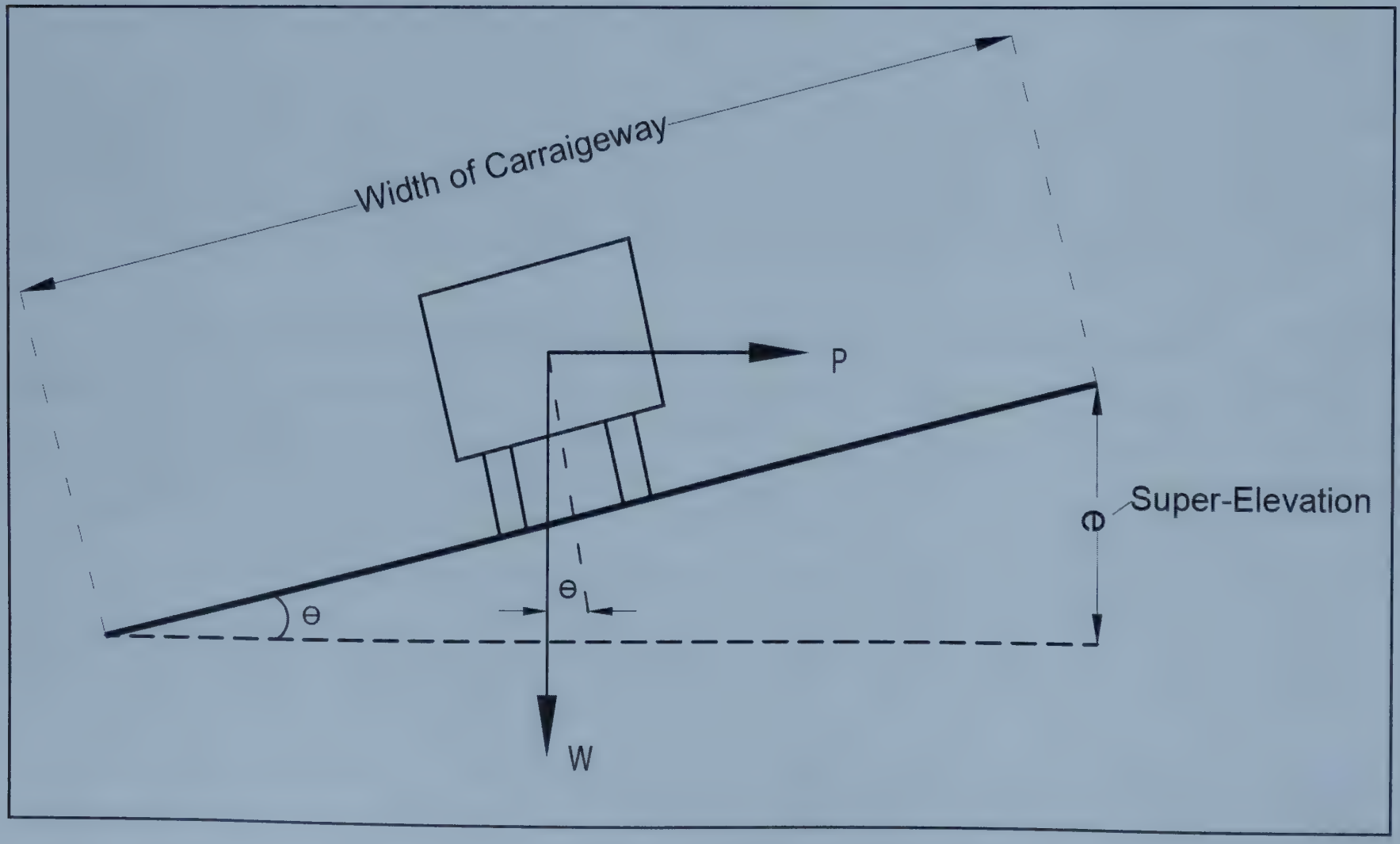


Fig. 6.2 Super-Elevated Pavement Section

6.4.3 *Maximum allowable Super-Elevation*

In the case of heavily loaded bullock carts or slower moving vehicles, the center of gravity of the vehicle will be relatively high and it will not be safe for such vehicles to move on the road with a high rate of super-elevation. Because of the slow speed, the centrifugal force will be negligibly small. Hence to avoid the danger of toppling of such loaded slow-moving vehicles, it is essential to limit the value of maximum allowable super-elevation. For 2/4/6/8 lane highways, super-elevation obtained from the above **equation 6.5** shall be limited to 7 percent, if radius of curve is less than the desirable minimum radius. It shall be limited to 5 percent if radius is more than or equal to the desirable minimum as well as at sections where project highway passes through an urban section or falls on a major junction. In case of MDR/ODR/VR, maximum limit of super-elevation is as given in the **Table 6.3** below. **Fig. 6.3** indicates the super-elevation for various design speeds on this basis.

Table 6.3 Maximum allowable Super-Elevation for MDR/ODR/VR

Nature of Terrain	Maximum allowable Super-Elevation
Plain and rolling terrain	7 percent
Snow-bound areas	7 percent
Hilly areas not bounded by snow	10 per cent

6.4.4 *Radii of Curves beyond which no Super-Elevation is required*

When the value of super-elevation obtained using the above **equation 6.5** is less than the road camber, the normal cambered section should be continued on the curved portion without providing any super-elevation.

If the horizontal curve radii obtained from the **equation 6.5** given in **section 6.4.2** (where 'e' is the camber in the straight section for the type of surface concerned) are greater than the values shown in the **Table 6.4**, then super-elevation is not required.

Table 6.4 Radii beyond which Super-Elevation/Transition is not required

Design Speed (kmph)	Radius (metres) for Camber of				
	4 percent	3 percent	2.5 percent	2.0 percent	1.7 percent
20	50	60	70	90	100
25	70	90	110	140	160
30	100	130	160	200	240
35	140	180	220	270	320
40	180	240	280	360	420
50	280	370	450	560	650
65	470	630	750	940	1100
80	710	950	1140	1420	1670
100	1110	1480	1780	2220	2610

6.4.5 *Methods of Attaining Super-Elevation*

The normal cambered section of the road is changed into super-elevated section in two stages.

First stage is the removal of adverse camber in outer half of the pavement. In the second stage, super-elevation is gradually built up over the full width of the carriageway (including the paved shoulders, if provided) so that required super-elevation is available at the beginning of the circular curve.

There are three different methods for attaining the super-elevation:

(i) Revolving pavement about the centre line.

(ii) Revolving pavement about the inner edge.

(iii) Revolving pavement about the outer edge.

Fig. 6.4 illustrates these methods diagrammatically. The small cross sections at the bottom of each diagram indicate the pavement cross slope condition at different points.

Each of the above methods is applicable under different conditions. Method (i) which involves least change in the profile of the carriageway will be found suitable in most of the situations where there are no physical controls and may be adopted in the normal course. Method (ii) is preferable where the lower edge profile is a major control, e.g. on account of drainage. Where overall appearance is the criterion, method (iii) is preferable since the outer edge profile which is mostly noticeable to drivers is not distorted.

The super-elevation should be attained gradually over the full length of the transition curve so that the design super-elevation is available at the starting point of the circular portion. The super-elevation transition section comprises of the super-elevation runoff and tangent runout sections. The super-elevation runoff section is the length of roadway needed to accomplish a change in outside-lane cross slope from zero (flat) to full super-elevation, or vice versa. The tangent runout section is the length of roadway needed to accomplish a change in outside-lane cross slope from the normal cross slope rate to zero (flat), or vice versa. In cases where transition curve is not provided for some reason, two-third super-elevation may be attained on the straight section before start of the circular curve and the balance one-third on the curve.

In developing the required super-elevation, it should be ensured that the longitudinal slope of the pavement edge compared to the centre-line (i.e. the rate of change of super-elevation) is not steeper than 1 in 150 for roads in plain and rolling terrain, and 1 in 60 in mountainous and steep terrain. **Table 6.5** mentions the minimum rate of change of super-elevation to be adopted for different type of terrains.

Table 6.5 Minimum Rate of Change of Super-Elevation

Type of Terrain	Minimum Rate of Change of Super-Elevation
	For 2/4/6/8 Lane Highways
Plain and Rolling	1 in 150
Mountainous and Steep	1 in 60

When cross-drainage structures fall on a horizontal curve, their deck should be super-elevated in the same manner as described above.

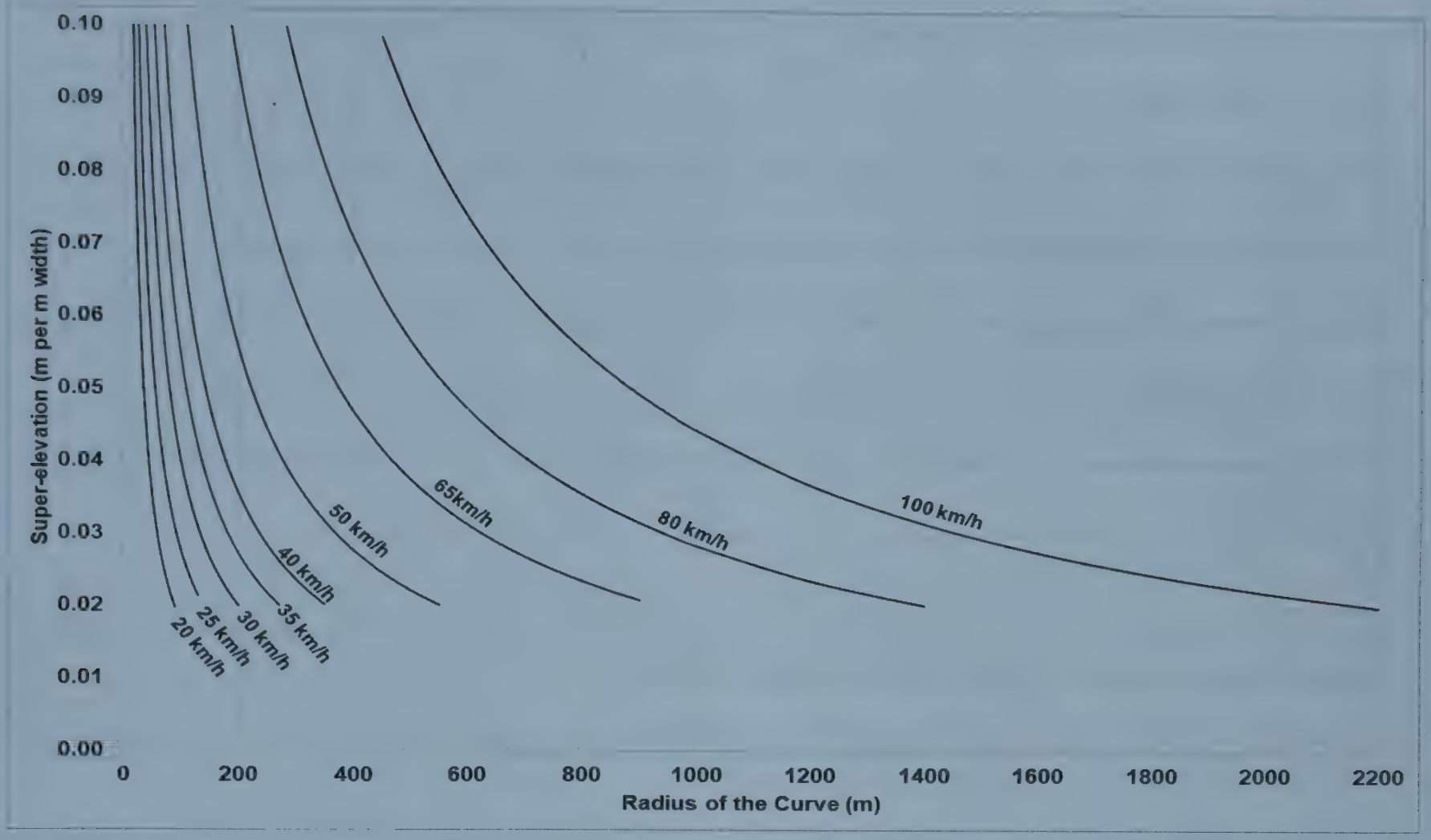


Fig. 6.3 Super-Elevation Rates for various Design Speeds

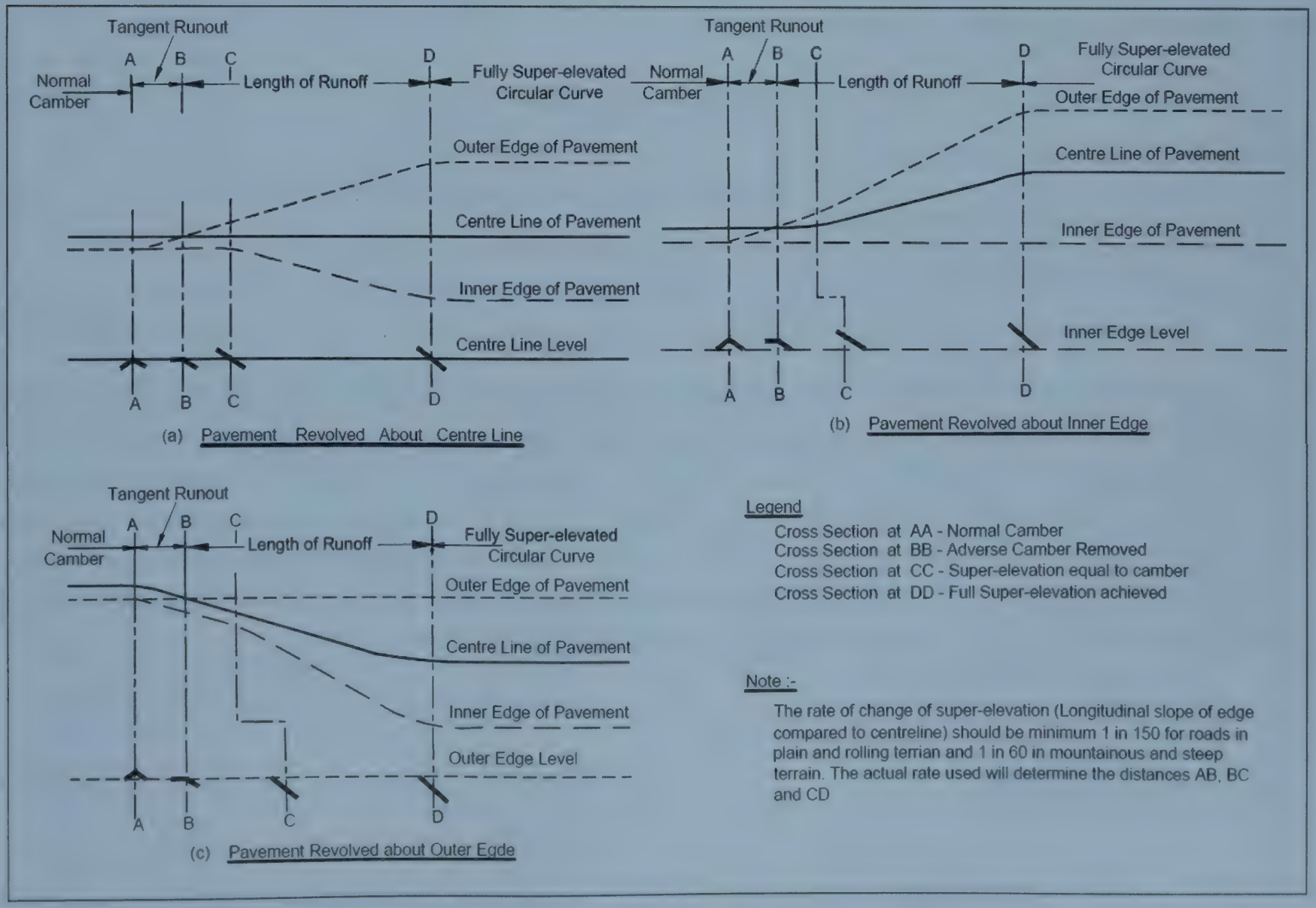

Tangent Runout
Normal Camber
A
B
C
D
Length of Runoff
Fully Super-elevated Circular Curve
Outer Edge of Pavement
Centre Line of Pavement
Inner Edge of Pavement
Centre Line Level
(a) Pavement Revolved About Centre Line
Inner Edge Level
(b) Pavement Revolved about Inner Edge
Outer Edge Level
(c) Pavement Revolved about Outer Egde
Legend
Cross Section at AA - Normal Camber
Cross Section at BB - Adverse Camber Removed
Cross Section at CC - Super-elevation equal to camber
Cross Section at DD - Full Super-elevation achieved
Note :-
The rate of change of super-elevation (Longitudinal slope of edge compared to centreline) should be minimum 1 in 150 for roads in plain and rolling terrian and 1 in 60 in mountainous and steep terrain. The actual rate used will determine the distances AB, BC and CD

Fig. 6.4 Schematic Diagrams showing different Methods of Attaining Super-Elevation

6.5 Extra Widening on Curves

6.5.1 *Necessity*

When vehicles negotiate a curve, the rear wheels generally do not follow the same track as that of front wheels. When the curve is not super-elevated the rear wheels follow the inner path on the curve as compared with those of the corresponding front wheels. On super-elevated curves, the relative position of the wheel path depends upon the speed and consequently upon the amount of friction developed for equilibrium. As the speed increases, the rear wheels assume position farther out. So, with higher speeds the path traced by rear wheels may fall outside as compared with those of the corresponding front wheels. Therefore, extra widening of the pavement is necessary to provide for this change in the overall track width required for travel at various speeds.

6.5.2 *Amount of Widening*

At sharp horizontal curves, it is necessary to widen the carriageway to provide safe passage of vehicles. The required widening has two components:

(i) Mechanical widening to compensate the extra width occupied by a vehicle on the curve due to off-tracking phenomenon of the rear wheels.

(ii) Psychological widening to permit easy crossing of vehicles since vehicles in a lane tend to deviate more on a curve than on a straight reach.

The formula for extra width on curves is given by

$$W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} \qquad \ldots (6.6)$$

Where, n = No. of traffic lanes

V = Design speed, kmph

R = Radius of horizontal curve, metre

l = Length of wheel base of longest vehicle (metres)

On two-lane or wider roads, it is necessary that both the above components should be fully catered for so that the lateral clearance between vehicles on curves is maintained equal to the clearance available on straights. Position of single-lane roads however is somewhat different. On single-lane roads, during crossing manoeuvres, outer wheels of vehicles should in any case use the shoulders on both the straight and the curve sections. It is therefore sufficient to consider only the mechanical component of widening on single-lane roads. **Fig. 6.5** shows the off-tracking phenomenon of vehicles and extra widening required at horizontal curves.

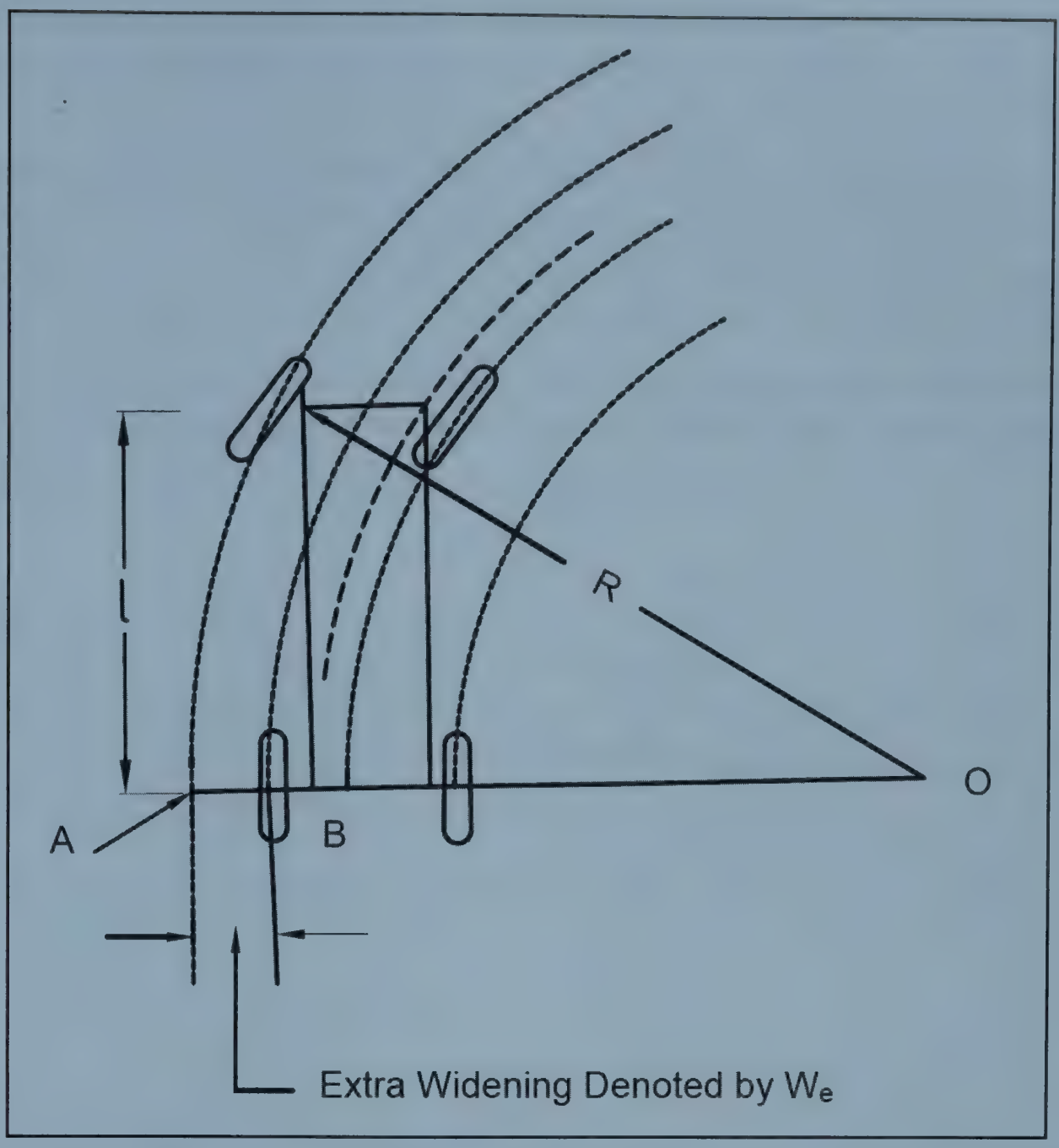


Fig. 6.5 Extra Widening provided at Horizontal Curves due to Off-Tracking

On horizontal curves of 2/4/6/8 lane highways with radius up to 300 m, width of pavement and roadway shall be increased as per **Table 6.6**. The extra width of carriageway to be provided at horizontal curves on single and two-lane roads for MDR/ODR/VR is given in **Table 6.7**.

Table 6.6 Extra Width of Pavement and Roadway for 2/4/6/8 Lane Highways

Radius of Curve	Extra Width
75-100 m	0.9 m
101-300 m	0.6 m

Table 6.7 Extra Width of Pavement at Horizontal Curves for MDR/ODR/VR

Radius of Curve (m)	Up to 20	21-40	41-60	61-100	101-300
Two-lane	1.5	1.5	1.2	0.9	0.6
Single lane	0.9	0.6	0.6	Nil	Nil

6.5.3 *Method of Widening*

The widening should be done by increasing the width at an approximately uniform rate along the transition curve. The extra width should be continued over the full length of the circular curve.

On curves having no transition, widening should be achieved in the same way as the super-elevation i.e. two-third being attained on the straight section before start of the curve and one-third on the curve.

The widening should be applied equally on both sides of the carriageway as shown in **Fig. 6.6**, except that on hill roads it will be preferable if the entire widening is done only on the hill side and not valley side due to physiological effect as shown in **Fig. 6.7**. Similarly, the widening should be provided only on the inside when the curve is plain circular and has no transition.

The extra widening may be attained by means of offsets radial to the centre line. It should be ensured that the pavement edge lines are smooth and there is no apparent kink.

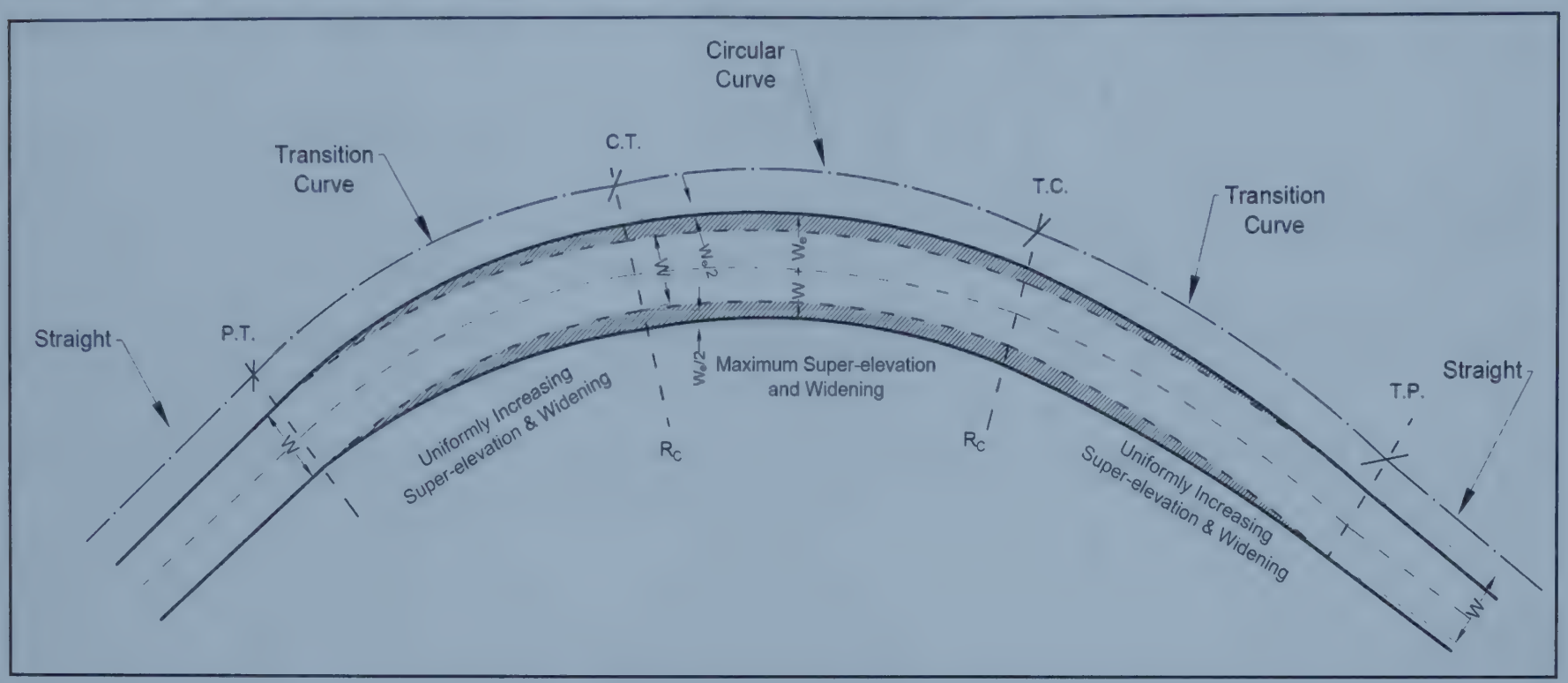


Fig. 6.6 Extra Widening on Horizontal Curve (Widening Distributed Equally on Both Sides of Central Line)

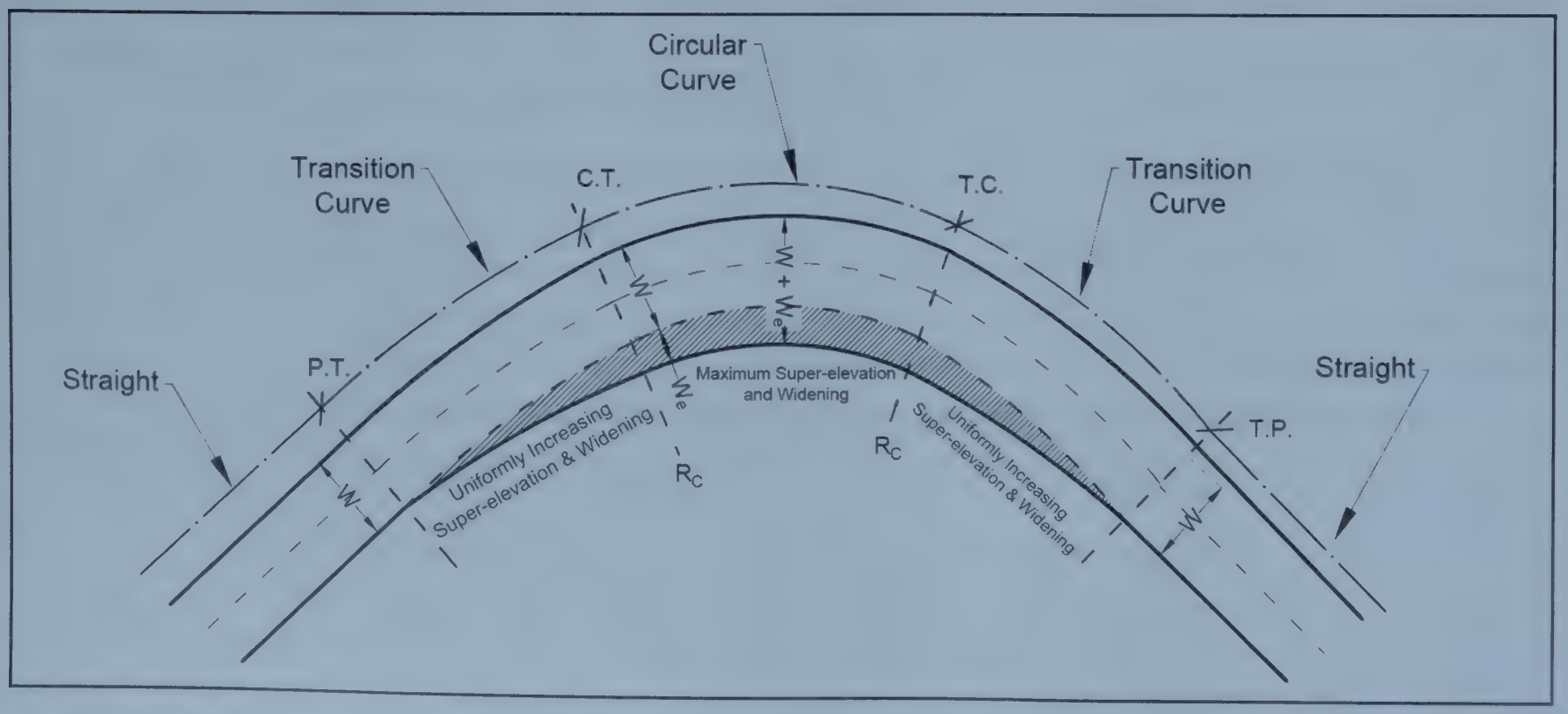


Fig. 6.7 Extra Widening on Sharp Horizontal Curve (Widening Distributed Entirely on Inside of Central Line)

6.6 Setback Distance on Curves

Sight distance across the inside of horizontal curves is an important element of design. Lack of visibility in the lateral direction may arise due to obstructions like walls, cut slopes, buildings, wooded areas, high farm crops, median plantation, etc. The straightforward manner of achieving the necessary setback in these situations is to remove the obstruction. If somehow this is not feasible, alignment of the road may need adjustments. Preferably each such case should be studied separately to determine the best course to adopt.

The setback distance to give the desired sight distance on the inside of horizontal curves can be calculated from the following equations (see **Fig. 6.8** for definitions):

(a) When length of curve > Stopping Sight Distance (SSD)

$$m=R-(R-n)\cos\theta \qquad \ldots (6.7)$$

where θ=half Central angle= $\frac{S}{2\,(R-N)}$ radians

m = the minimum setback distance to sight obstruction in meters at the middle of the curve (measured from the center line of the road)

R = radius at the center line of the road in meters

n = distance between the center line of the road and the center line of the inside lane in meters

S = sight distance in meters

In the above equation, sight distance is measured along the middle of inner lane. On narrow, single-lane roads, this refinement is not necessary and the setback distance should be provided with respect to the center-line of the road, i.e., assuming 'n' to be zero.

(b) When length of curve < Stopping Sight Distance (SSD)

$$m' = R - (R-n)\,Cos\,\theta' + \frac{S-L_c}{2}\,Sin\,\theta' \qquad \ldots (6.8)$$

Where θ'=half Central angle = $\frac{L_c}{2\,(R-n)}$ radians

m' = the minimum setback distance to sight obstruction in meters at the middle of the curve (measured from the center line of the road)

R = radius at the center line of the road in meters

n = distance between the center line of the road and the center line of the inside lane in meters

S =sight distance in meters

L_c = Length of the horizontal curve

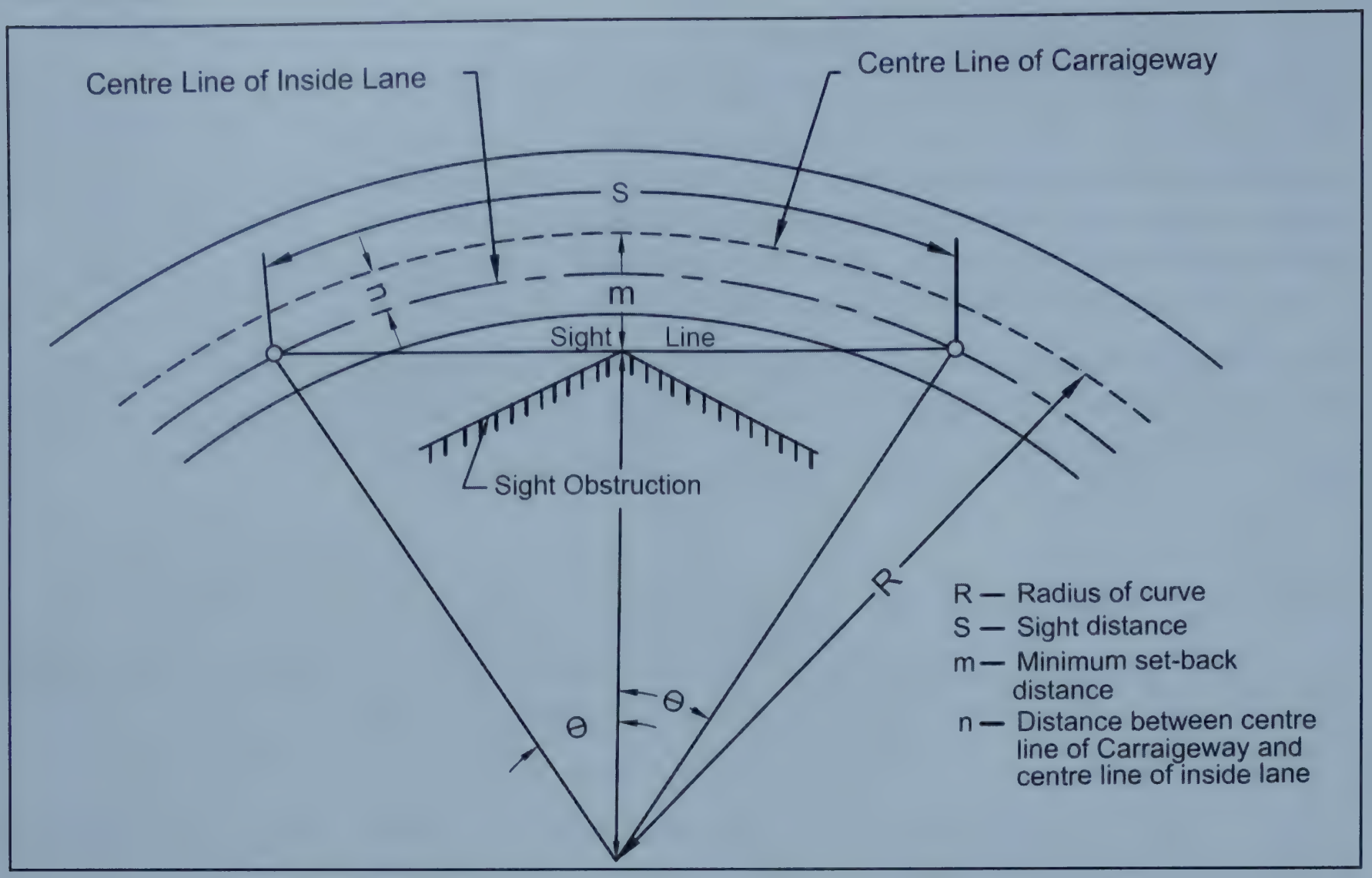


Fig. 6.8 Setback Distance at Horizontal Curves

Utilizing the above equation, design charts for lateral clearance corresponding to the safe stopping distance are given in **Fig. 6.9** for two-lane roads. The plotted values relate basically to circular curves longer than the design sight distance. For shorter curves, the values of setback distance found from **Fig. 6.9** will be somewhat on the higher side, but these can any way be used as a guide.

Lateral clearance/setback distance for overtaking or intermediate sight distance (desirable minimum sight distance) could be computed similarly. Calculations would, however, reveal that the setback distance required will usually be too large to be economically feasible except on very flat curves.

When there is a cut slope on the inside of the horizontal curve, a practical consideration in providing the setback distance is the average height of sight line above the ground level. For stopping sight distance, the average height can be assumed as 0.7 m, since the height criteria are 1.2 m for the eye and 0.15 m for the object. Cut slopes should be kept clear above this height at the midpoint of the sight line by cutting back the slope or benching. In the case of meeting or overtaking sight distance, height of sight line above the ground would be 1.2 m.

Where a horizontal and summit vertical curve overlap, the line of sight will not be over the top of the crest but to one side, and in part may be off the roadway. Design in such cases should provide for the required sight distance both in the vertical direction along the pavement and in the horizontal direction on the inside of the curve.

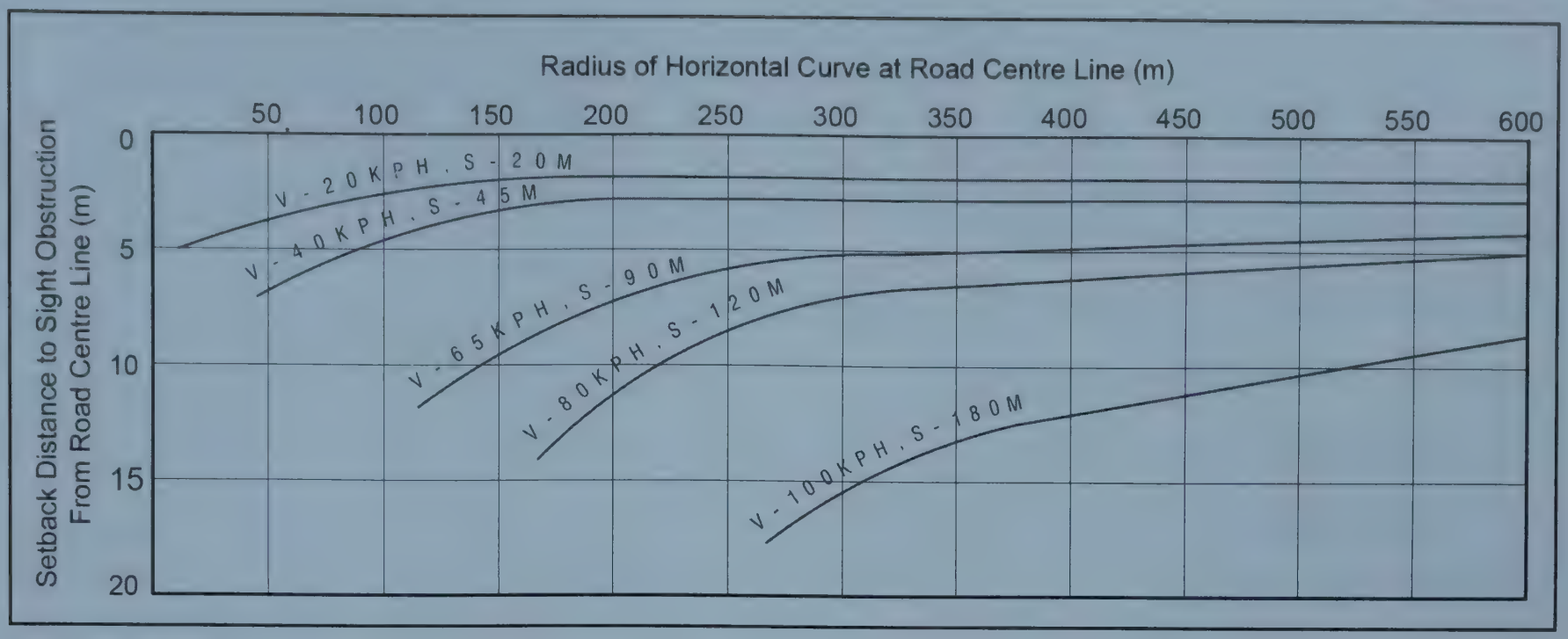


Fig. 6.9 Minimum Setback Distance Required at Horizontal Curves on Two Lane Roads for Safe Stopping Sight Distance

Example on Setback Distance

There is a horizontal curve of radius 400 m and length 200 m on a highway. Compute the set-back distances required from the center line on the inner side of the curve so as to provide for (a) stopping sight distance of 90 m (b) safe overtaking sight distance of 300 m. The distance between center lines of road and the inner lane is 1.9 m.

Solution:

Case (a): For stopping sight distance of 90 m

Given: R = Radius of the curve = 400 m

L_c= Length of the curve = 200 m

S =Stopping sight distance = 90 m

n = The distance between center lines of road and the inner lane = 1.9 m

Since, the stopping sight distance (S) < length of the curve

The set-back distance can be computed using following equation:

$$m = R-(R-n)\cos\theta$$

$$\theta = \text{half Central Angle} = \frac{S}{2\,(R-n)} \text{ radians}$$

$$\theta = \frac{180S}{2\pi\,(R-n)} \text{ Degrees}$$

$$\theta = \frac{180^{*}\,90}{2\pi\,(400-1.9)} \text{ Degrees}$$

$$\theta = 6^{\circ}\,29' \text{ Degrees}$$

Now,

$m = 400-(400-1.9)\cos 6° 29'$

$m = 4.4$ meters

Therefore, the set-back distance required from the center line of the road to provide stopping sight distance of 90 m is 4.4 meters.

Case (b): For safe overtaking sight distance of 300 m

Given: R = Radius of the curve = 400 m

L_c = Length of the curve = 200 m

S = Safe overtaking sight distance= 300 m

n = The distance between center lines of road and the inner lane = 1.9 m

Since, the safe overtaking sight distance (S) > length of the curve

The set-back distance can be computed using following equation:

$$m' = R-(R-n)\ \cos\theta' + \frac{S-L_c}{2}\ \sin\theta'$$

Where $\theta' = half\ Central\ Angle = \frac{L_c}{2(R-n)}\ radians$

$$\theta' = \frac{180\ L_c}{2\pi\ (R-n)}\ Degrees$$

$$\theta' = \frac{180^* 200}{2\pi\ (400-1.9)}\ Degrees$$

$$\theta' = 14.39\ Degrees$$

Now,

$$m' = 400-(400-1.9)\cos 14.39 + \frac{300-200}{2}\ \sin 14.39$$

$$m' = 26.8\ m$$

Therefore, the minimum set-back distance required from the center line of the roads on the inner side of pavement to provide an OSD of 300 m = 27 meters

6.7 Transition Curve

6.7.1 *Necessity of Transition Curve*

Transition curve has a radius which decreases from infinity at the tangent point to a designed radius of the circular curve. When a transition curve is introduced between straight and circular curve, the radius of the transition curve decreases, becomes equal to the radius of circular curve at start of the circular curve. The rate of change of radius of the transition curve will depend on the equation of the curve or its shape. Thus, transition curves are necessary for a vehicle to have smooth entry from a straight section into a circular curve at the design speed. Properly designed transition curves shall be provided at both ends of the circular curve. Transition curve serves the following functions in the horizontal alignment of highway:

(a) To introduce gradually the centrifugal force between the tangent point and the beginning of the circular curve, avoiding sudden jerk on the vehicle

(b) To enable the driver, turn the steering gradually for his own comfort and security

(c) To enable gradual introduction of the designed super-elevation and extra widening of pavement (if any) at the start of the circular curve

(d) To improve the aesthetic appearance of the road

6.7.2 *Types of Transition Curves*

The essential requirements of a transition curve are:

(a) Radius of curvature should decrease gradually from infinity to the minimum to enable the steering wheel to be turned gradually and to eliminate the shock due to application of centrifugal force.

(b) The rate of change of centrifugal acceleration, i.e., 'C' should be such that it does not cause discomfort or undesirable oscillation. Since the radial acceleration is inversely proportional to the radius of curvature for any fixed speed, the transition should be such that the radius of curvature should be inversely proportional to the length of the curve from the starting point.

Fulfilling the above criteria, commonly 3 types of transition curves are adopted in horizontal alignment

(i) Spiral (also called Clothoid)

(ii) Lemniscate

(iii) Cubic Parabola

6.7.3 *The Spiral*

In general, horizontal curves should consist of either a circular curve flanked by spiral transitions at both ends or two transition spiral curves connected to each other. Spiral curves are recommended for transition since the spiral curve satisfies the requirements of an ideal shape of transition curve and the geometric property of spiral is such that the calculations and setting out the curve in the field are simple and easy.

An ideal transition curve is a spiral, which has the property that product of the length of spiral (from the starting point to any point on it) and the radius of curvature of the point under consideration is constant.

$$L.R = L_S R_C = constant$$

Where, L_S = Length of the spiral from the starting point to any point on it

R_C = Radius of curvature of the point under consideration

6.7.4 *Minimum Length of Transition Curves*

Minimum length of the transition curve for single lane, intermediate lane & multi-lane roads should be determined from the following two considerations and the larger of the two values should be adopted for design.

(i) The rate of change of centrifugal acceleration should not cause discomfort to drivers. From this consideration, the length of transition curve is given by:

$$L_s = \frac{0.0215V^3}{CR} \quad \ldots (6.9)$$

where

L_s = length of transition curve in metres

V = Design speed in kmph

R = Radius of circular curve in metres

$C = \frac{80}{75 + V}$ (allowable rate of change of centrifugal acceleration, $\frac{m}{sec^3}$ subject to a maximum of 0.8 and minimum of 0.5)

(ii) The rate of change of super-elevation (i.e. the longitudinal grade developed at the paved shoulder edge compared to through grade along the centre line) should be such that it does not cause discomfort to travellers or to make the road appear unsightly. For 2/4/6/8 lane highways, rate of change should not be steeper than 1 in 150 for roads in plain and rolling terrain, and 1 in 60 in mountainous/steep terrain. From this consideration, the length of transition curve is given by:

(a) If pavement is rotated about the centre line

$$L_s = \frac{EN}{2} = \frac{eN}{2}(W+W_e) \quad \ldots (6.10)$$

(b) If pavement is rotated about the inner edge

$$L_s = EN = eN(W+W_e) \quad \ldots (6.11)$$

where

L_s = Length of transition curve in metres

E = Total raise of the pavement with respect to the inner edge

e = Rate of super-elevation

1 in N = Rate of change of super elevation

W = Normal pavement width in metres

W_e = Extra widening provided at the circular curve

The minimum length of transition curves for expressways should as per IRC:SP:99.

Example on Length of Transition Curve

Calculate the length of transition curve for a two-lane highway for minimum design speed of 80 kmph. Radius of circular curve= 250 m. The allowable rate of introduction of super-elevation (pavement rotated about the inner edge) = 1 in 150

Solution:

(a) Length of the transition curve as per the rate of change of centrifugal acceleration

Allowable rate of change of centrifugal acceleration, $C = \frac{80}{(75 + V)} = \frac{80}{(75 + 80)} = 0.52\ m/sec^3$

$$L_s = \frac{0.0215V^3}{CR} = \frac{0.0215*80^3}{0.52*250} = 85\ m$$

(b) Length of the transition curve as per the rate of change of super-elevation

$$e = \frac{V^2}{225R} = \frac{80^2}{225*250} = 0.11$$

As this value is greater than maximum allowable rate of 0.07, limit the value of 'e' to 0.07. Check the safety against the transverse skidding for the design speed of 80 kmph

$$f = \frac{V^2}{127R} - e = \frac{80^2}{127*250} - 0.07 = 0.20 - 0.07 = 0.13$$

As this value of f is less than the allowable value of 0.15, the super-elevation rate of 0.07 is safe for design speed of 80 kmph

Total width of two-lane carriageway= 3.5*2 + 1.5*2 = 10 m (For a two-lane highway, 3.5 m as width per lane and 1.5 m of paved shoulder on either side)

Extra widening at horizontal curve for a radius of 250 m= 0.6 m as per **Table 6.6**

Total width of two-lane carriageway at the curve $(W+W_e)$= 10 +0.6 = 10.6 m

If pavement is rotated about the inner edge,

$$L_s = EN = eN(W+W_e) = 0.07*150*10.6 = 111\ m$$

Adopting the highest value of the two criteria, i.e. 111 m as the design length of the transition curve

6.7.5 *Radii beyond which no Transition is required*

Centrifugal ratio is inversely proportional to the radius of curve. Therefore so for curves of large radii, the centrifugal force developed is small. Radii beyond which transition is not required is specified in **Table 6.4**.

6.7.6 *Design of Curves with Transition*

In designing a road alignment, the angle of deviation in the alignment is known. The classification of the highway in which the curve is to be located is also known. If the terrain is one where there are no features which will restrict the alignment, it is desirable that the maximum possible radius be selected which is also known. The minimum transition lengths, L_s for different speeds and radii are also known.

If two transition curves of equal lengths are adopted on both sides of the central circular arc, the angle subtended by circular curve is given below:

Total deviation angle = Central angle of circular Arc + 2 Deviation angle of transition curve*

$$\Delta=\Delta_c+2*\theta_s \quad \text{... (6.12)}$$

$$\theta_s=\frac{L_s}{R_c} \quad \text{... (6.13)}$$

Length of circular curve is given by

$$L_c=R*\Delta_c \quad \text{... (6.14)}$$

Where Δ_c is in radians

6.7.7 *Setting out of Spiral Transition*

From the design of the curve, T_s, the tangent distance, E_s, the apex distance, L_s, length of the transition, R_c, the radius of the circular curve, and θ_s, the tangent deviation of the transition, etc., are known. The start of the curve, TP is fixed by measuring the T_s from the apex point of the curve which is fixed in the alignment. Where the apex point, HIP, is not accessible, TP, the start of the curve may be established with reference to known objects shown in the plans, by sealing the distances. The curve is laid out in the field by any one of the following methods:

- ✓ Off-sets from the tangent method
- ✓ Off-sets 1/4-point method
- ✓ Off-sets from the tangent and the circular method
- ✓ Polar deflection method

Due to the introduction of transition curve between tangent and circular curve, there is a shift of the transition curve given by

$$S=\frac{L^2_s}{24R_c} \quad \text{... (6.15)}$$

6.7.8 *Design of Curves in Tight Locations*

Whenever a curve has to be designed, the controlling factors in the field are Δ, the deviation angle, T_s, the tangent distance, and E_s, the apex distance. The design requirements are the minimum radius of the curve and the minimum length of the transition for various design speeds. Δ is fixed by the alignment and T_s and E_s distances suitable to the curve depend upon the topographical conditions at the curve. For example, there may be the situations where the E_s or T_s distances have to be equal to a specific value or when the curve has to pass through a predetermined point or when the shift of the curve has to be exactly equal to a specific value. Under such conditions, the curve will have to be designed to suit the field limitations. Such a case will generally arise when introducing transitions on existing roads with circular curves.

The following are some of the methods by which transitions could be introduced into existing circular curve alignments.

- The radius of curve is reduced by an amount equal to the shift of the proposed transition and the transition introduced. This will necessitate no change in alignment of the straight portions

- The radius of the original curve is retained and its centre shifted by an amount equal to $\frac{S}{Cos\left(\frac{\Delta}{2}\right)}$ thus providing the necessary space for the introduction of the transition. Here also the straight portions are not to be shifted.
- The original circular curve is retained and the straights shifted by an amount equal to the shift which is required for accommodating the transition. It is necessary that the effect of the shifting of the tangents on the adjoining curves be considered before adopting this method.

6.8 Transitions for Compound Curves

Transitions are used between curves of different radii to change gradually from one circular motion to another of greater or less degree and to permit a corresponding change in super-elevation. The sharper curve must necessarily lie inside the flatter curve. **Fig. 6.10** shows a typical compound curve with two different radii.

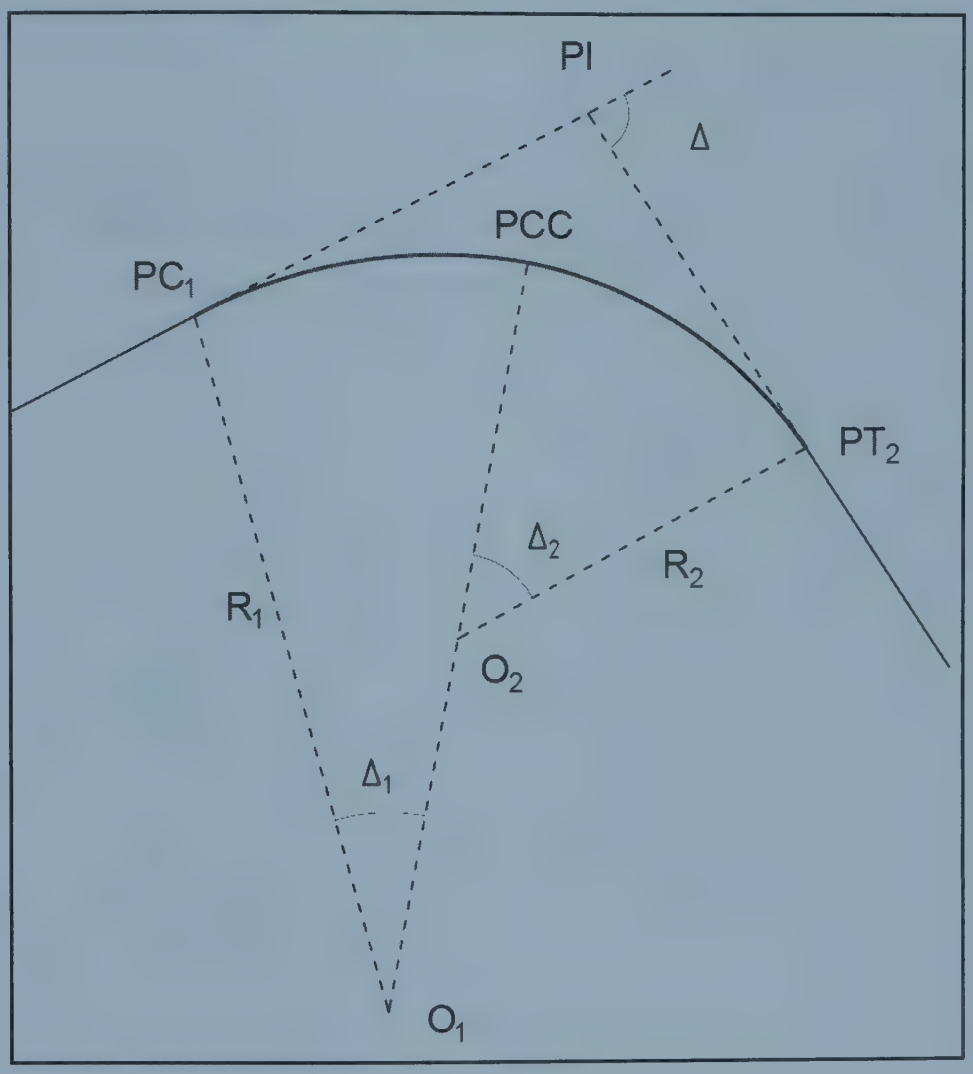


Fig. 6.10 Compound Curve

The length of transition required is the difference between the lengths required for changing from a longitudinal to a circular motion for each curve so that

$$L_S = L_{S2} - L_{S1}$$

$$L_S = \frac{0.215V^3}{C * R_2} - \frac{0.215V^3}{C * R_1} \quad \ldots (6.16)$$

To change gradually from the super-elevation of one curve to that of another, the slope of the outer edge of pavement with respect to the centre line should not be steeper than 1 in 150 so that the minimum length of transition should be at least 150 times the difference between the full super-elevation of the two curves.

Compounding two curves that differ in radius by more than 50 per cent of the smaller radius results in a sudden change of curvature which is noticeable if the transition is omitted. A transition of at least 30 m long should be inserted. Where two curves differ in radius by less than 50 per cent, and the preceding methods of design result in transitions less than 30 m long, they may be omitted and such change in super-elevation required can be provided on the flatter curve.

Example

Determine the length of transition required between the two curves of radii 600 m and 250 m for design speed of 80 km/h.

C=250 m = 80/(75+80) = 0.5161

$1/ R_a = 1/250 – 1/600 = 0.00233$

$L_s = (0.0215*80*80*80*0.00233)/0.5161 = 49.69$ say 50 m

6.9 Transitions for Reverse Curves

It is of great importance that transition curves should be inserted when two curves of opposite direction are in close proximity to one another. **Fig. 6.11** shows a typical reverse curve.

The length of transition required is the sum of the lengths required for reverse curves, so that

$$L_S = L_{S2} + L_{S1}$$

$$L_S = \frac{0.215V^3}{C * R_2} + \frac{0.215V^3}{C * R_1} \quad \ldots (6.17)$$

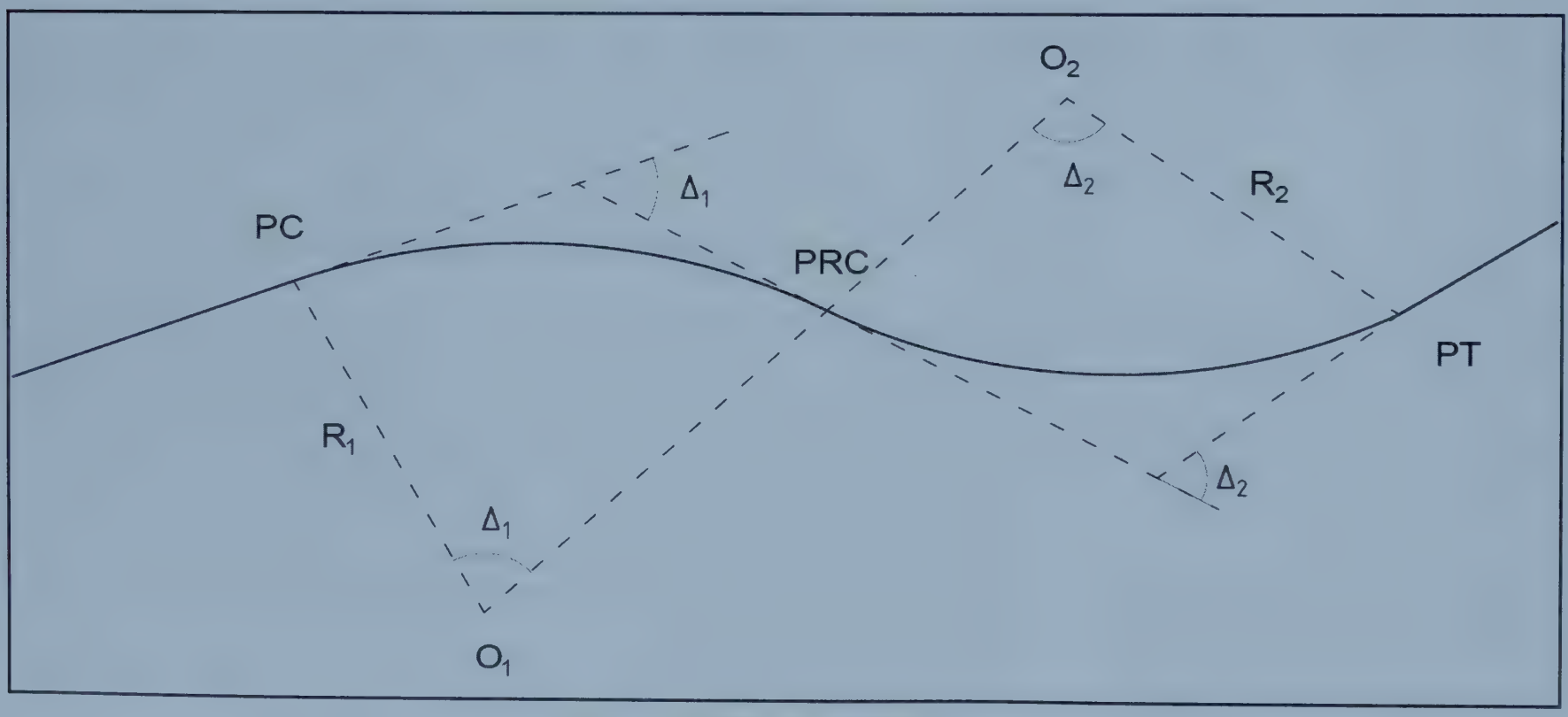


Fig. 6.11 Reverse Curve

6.10 Transitions for Hair Pin Bends

A hair-pin bend may be designed as a circular curve with transition curves at each end. Alternatively, compound circular curves may be provided. **Fig. 6.12** shows a hair pin bend provided with a compound circular curve.

The following design criteria should be adopted normally for the design of hair-pin bends (IRC: 52):

a. Minimum design speed - 20 km/h

b. Minimum roadway width at apex

   i. National/State Highways – 11.5 m for double-lane, 9.0 m for single-lane

   ii. Major District Roads and Other District Roads - 7.5 m

   iii. Village Roads – 6.5 m

c. Minimum radius for the inner curve - 14.0 m

d. Minimum length of the transition –15.0 m

e. Gradient

   i. Maximum - 1 in 40 (2.5%)

   ii. Minimum – 1 in 200 (0.5 %)

f. Super-elevation – 1 in 10 (10 %)

Inner and outer edges of the roadway should be concentric with respect to centre line of the pavement.

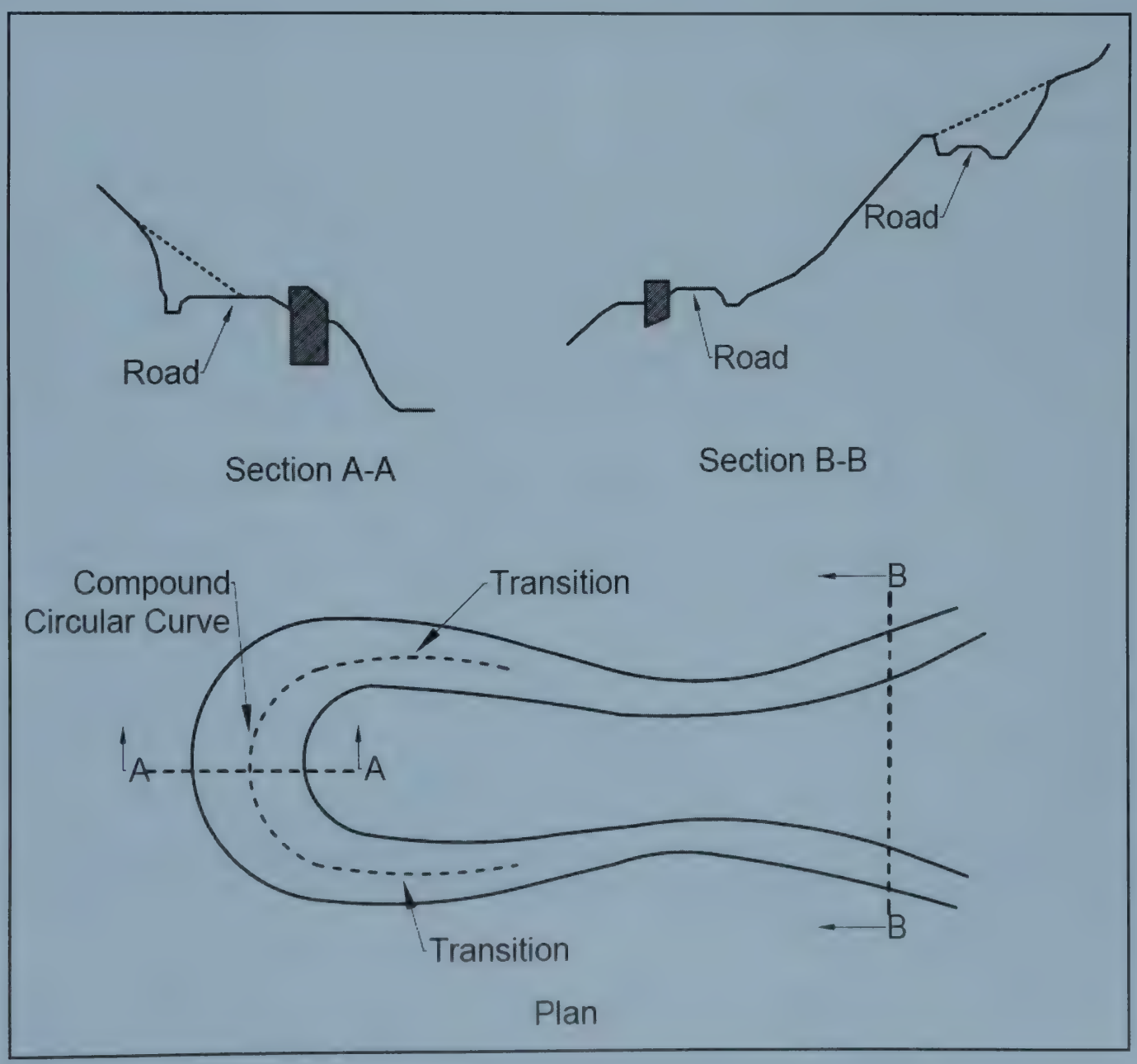


Fig. 6.12 Hair Pin Bend with a Compound Circular Curve

Where a number of hair-pin bends have to be introduced, a minimum straight length of 60 m should be provided between the successive bends to enable the driver to negotiate the alignment smoothly.

Widening of hair-pin bends at a later date is a difficult and costly process. Moreover, gradients tend to become sharper, as widening can be generally achieved only by cutting the hill side.

These points should be kept in view at the planning stage, specially where a series of hair-pin bends are involved. At hair-pin bends, preferably the full roadway width should be surfaced.

7. VERTICAL ALIGNMENT

7.1 Introduction

The need of providing vertical curves arises due to the fact that the alignment should follow the terrain. The terrain is not always flat. Undulations, if small, can be rectified to make the surface flat. But this may not be possible if the variations in surface levels are large. In such cases, it is economical to follow the terrain. The tangents drawn at different locations define and fix the location of a vertical curve to be provided. These tangents may fall or rise along the alignment and hence are termed as grades. The amount of fall and rise has to be fixed while considering the performance of the vehicle expected to use that alignment. The provision of vertical curves is also required to provide smooth and safe movement of vehicles along the longitudinal profile. These needs have increased in the light of technological advancement in vehicles i.e. steering, braking, speeding, etc.

7.2 Deviation Angle and Gradients

Gradient is defined as the rate of rise or fall along the longitudinal profile of the road. This rise or fall can be reported as 'unit per length' or as 'percentage' or in a form of 'an angle of inclination'. These grade lines will act as a tangent along the profile until there is a change in the grade line. At this point the angle of movement changes. This change in angle or angle between the two intersecting grade lines is known as 'deviation angle' or 'angle of intersection'. This is computed as algebraic difference between the first grade and second grade taken in order. This is shown in **Fig. 7.1**.

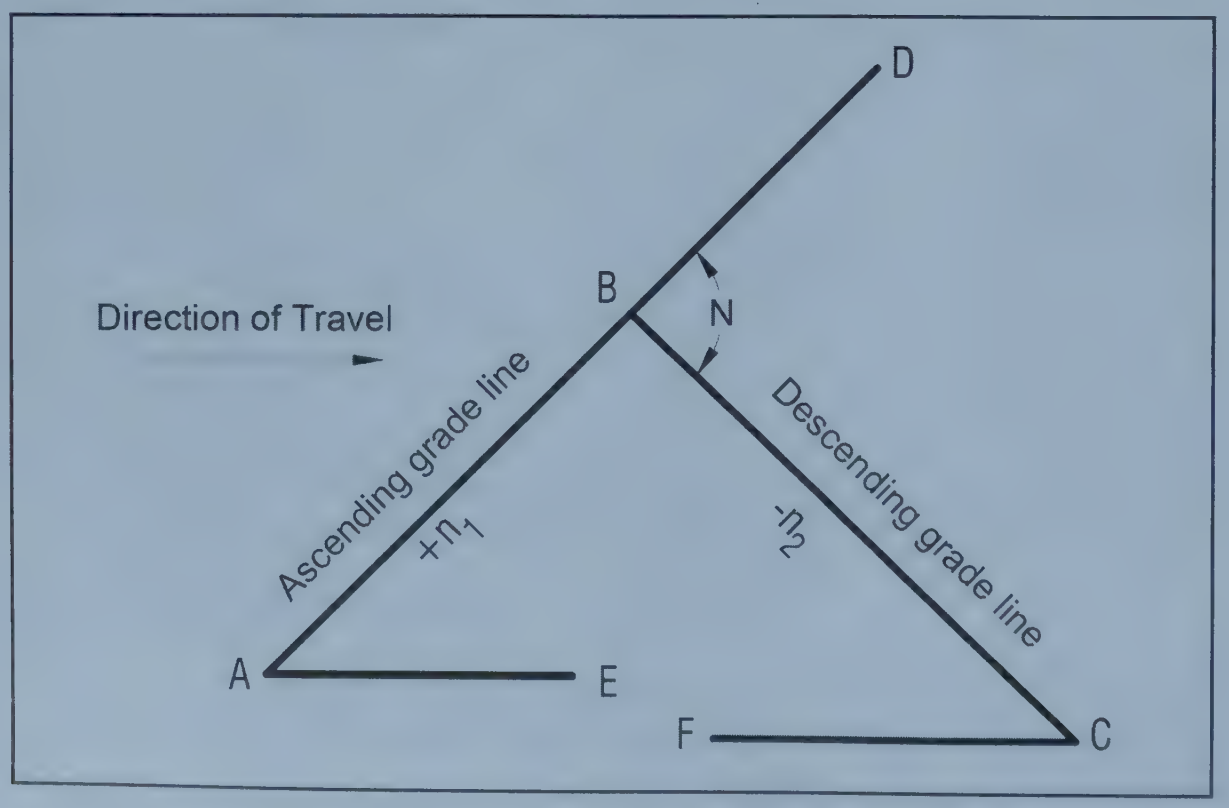


Fig. 7.1 Grades and Deviation Angle

Fig. 7.1 shows two grade lines with upward slope of n_1% and downward slope of n_2%. The movement is from upward slope to downward slope. In such a case the deviation angle, shown as 'N' will be:

$$N = (n_1-(-n_2))\% \quad \ldots (7.1)$$

If the slope of the grade line is represented as 1 in n_1 and 1 in n_2 then:

$$N = \frac{1}{n_1} + \frac{1}{n_2} \quad \ldots (7.2)$$

Example 1:

An upgrade of 3% is followed by a downgrade of 2%. The deviation angle would be:

$$N = +3 - (-2) = 5\%$$

Example 2:

A downgrade of 1 in 100 is followed by an upgrade of 1 in 200. The deviation angle would be:

$$N = (-1/100) - (1/200) = -3/200 = -1.5\%$$

Gradients must be fixed before a vertical curve can be designed. The designer should always keep an eye on economy in selecting the alignment and suggesting the longitudinal profile of a road. The primary concern in selecting a gradient is the ease and convenience of driving as well as the acceptable reduction in the performance of the vehicle while negotiating the gradient. In hilly regions, the gradients should be such that they can be negotiated with the least change of gears by the heavier vehicles. This saves time and operational costs.

Recommended gradients for different classes of terrain should be as given in **Table 7.1** for highways.**Table 7.2** includes the recommended gradients in different terrains for MDR/ODR/VR. Although the new vehicles are able to easily negotiate the steep gradients, the gradients given in **Table 7.1** and **Table 7.2** have been kept in such a way that even old heavy vehicles should be able to negotiate the gradients with ease.

Table 7.1 Recommended Gradients for 2/4/6/8-Lane Highways in different Terrains

S. No.	Terrain	Ruling Gradient	Limiting Gradient
1.	Plain or rolling	2.5 per cent (1 in 40)	3.3 per cent (1 in 30)
2.	Mountainous	5 per cent (1 in 20)	6 per cent (1 in 16.7)
3.	Steep	6 per cent (1 in 16.7)	7 per cent (1 in 14.3)

Note: *For expressways, the ruling gradient shall be 2%. Ruling gradient shall be adopted for expressways as far as possible. Limiting gradient (3%) shall be adopted only in very difficult situations and for short lengths.*

Table 7.2 Recommended Gradients for MDR/ODR/VR in different Terrains

S. No.	Terrain	Ruling Gradient	Limiting Gradient	Exceptional Gradient
1.	Plain or rolling	3.3 per cent (1 in 30)	5 per cent (1 in 20)	6 per cent (1 in 16.7)
2.	Mountainous terrain, and steep terrain having elevation more than 3,000 m above the mean sea level	5 per cent (1 in 20)	7 per cent (1 in 14.3)	10 per cent (1 in 10)
3.	Steep terrain up to 3,000 m height above mean sea level	6 per cent (1 in 16.7)	8 per cent (1 in 12.5)	10 per cent (1 in 10)

Gradients up to the 'ruling gradient' may be used as a matter of course in design of roads, grade separators and ROBs. However, in special situations such as isolated over- bridges in flat country or roads carrying a large volume of slow-moving traffic, it is recommended to use a flatter gradient of 2 percent from the angle of aesthetics, traffic operations, and safety.

The 'limiting gradients' may be used where the topography of a place compels this course or where the adoption of gentler gradients would add enormously to the cost. In such cases, the length of grade, steeper than the ruling gradient should be as short as possible.

'Exceptional gradients' are meant to be adopted only in very difficult situations and for short lengths not exceeding 100 m at a stretch. In mountainous and steep terrain, successive stretches of exceptional gradients must be separated by a minimum length of 100 m having gentler gradient (i.e. limiting gradient or flatter).

The rise in elevation over a length of 2 km should not exceed 100 m in mountainous terrain, 120 m in steep terrain and an additional lane is required for longer length in case of a steep up-gradient.

Minimum Gradients for Drainage: On non-kerbed pavements in embankment, near-level grades are not objectionable when the pavement has sufficient camber to drain the storm water laterally. However, in cut sections or where the pavement is provided with kerbs, it is necessary that the road should have some gradient for efficient drainage. Desirable minimum gradient for this purpose should be considered as per IRC:SP:42 "Guidelines on Road Drainage" and IRC:SP:50 "Guidelines on Urban Drainage".

7.3 Practical Considerations

The following points of guidance will be helpful for the design of vertical curves on highways:

a. The vertical alignment should provide a smooth longitudinal profile and grade changes should not be too frequent as to cause kinks and visual discontinuities in the profile. Desirably, there should not be any change in gradient within a distance of 150 m. In mountainous terrain, a minimum distance of 75 m may be acceptable under constrained situations. In case of overlay sections, due to cost implications, the distance can be minimum of 90 m.

b. A short valley curve within a continuous profile is undesirable since this tends to distort the perspective view and can be hazardous.

c. Broken-back grade lines i.e. two vertical curves in the same direction separated by a short tangent (length less than the distance covered in 10 seconds at design speed), should be avoided due to poor appearance, and preferably replaced by a single long curve.

d. Decks of small cross-drainage structures, (i.e., culverts and minor bridges) should follow the same profile as the flanking road section, without any break in the grade line.

e. For small bridges up to 30 m span and having horizontal deck, it would be preferable to combine the flanking sections into a single vertical curve.

f. The overall appearance of a highway can be enhanced considerably by judicious combination of the horizontal and vertical alignments. Plan and profile of the road should not be designed independently so as to produce an appropriate three-dimensional effect. Proper coordination in this respect will ensure safety, improve utility of the highway and contribute to overall aesthetics.

g. The degree of curvature should be in proper balance with the gradients. Straight alignment or flat horizontal curves at the expense of steep or long grades, or excessive curvature in a road with flat grades, do not constitute balanced designs and should be avoided.

h. The vertical curvature of roads should be such that they take in all minor changes in ground levels.

i. As far as possible, numerous changes in gradients joined together with short vertical curves should be avoided, except in mountainous terrain where the adoption of long and easy curves might become very costly.

j. The economic aspect of vehicle operation is very important in the choice of grades since the greater consumption of fuel, the heavier wear and tear of tyres and brakes of vehicles in traversing a wide range of vertical rises and falls would add heavily to operation costs.

k. Vertical curvature superimposed upon horizontal curvature gives a pleasing effect. As such the vertical and horizontal curves should coincide as far as possible and their length should be more or less equal. If this is difficult for any reason, the horizontal curve should be somewhat longer than the vertical curve.

l. Sharp horizontal curves should be avoided at or near the apex of pronounced summit/sag vertical curves from safety considerations.

m. It is desirable that the deck or top level of culverts should be fixed in line with the grade line of the flanking sections of the road so that no hump is formed. This may not, however, be possible on an existing road where there are culverts with deck levels higher than the general road levels and the height of hump not sufficient enough to obstruct the sight line. For such cases, the approaches on either side should be provided with smooth vertical curves (summit and valley curves).

7.4 Types of Vertical Curves

The fixing of an alignment in a terrain or a topography may necessitate a change in direction of movement in the vertical profile. Undulations of the surface will govern the same if the size of an undulation is large. If it is small, then the surface can be graded or made flat. The combination of

two grades at a point of intersection may result in a concave or a convex curve. Based on these the vertical curves can be classified as:

a. Summit curves

b. Valley curves

When a vehicle approaches a summit curve, the view of the road is cut off beyond the summit. Therefore, to secure the required sight distance, the intersection of the two grades should be eased off by interposing a properly designed vertical curve. For valley curves, visibility is not a problem during day time. However, for night travel, the design must ensure that the roadway ahead is illuminated by vehicle headlights to a sufficient length enabling the vehicle to brake to a stop if necessary.

7.5 Summit Curves

7.5.1 *General*

Summit curves are identified by its convex shape. There are different combinations of grade lines which will result in a summit curve. These are shown in **Fig. 7.2**.

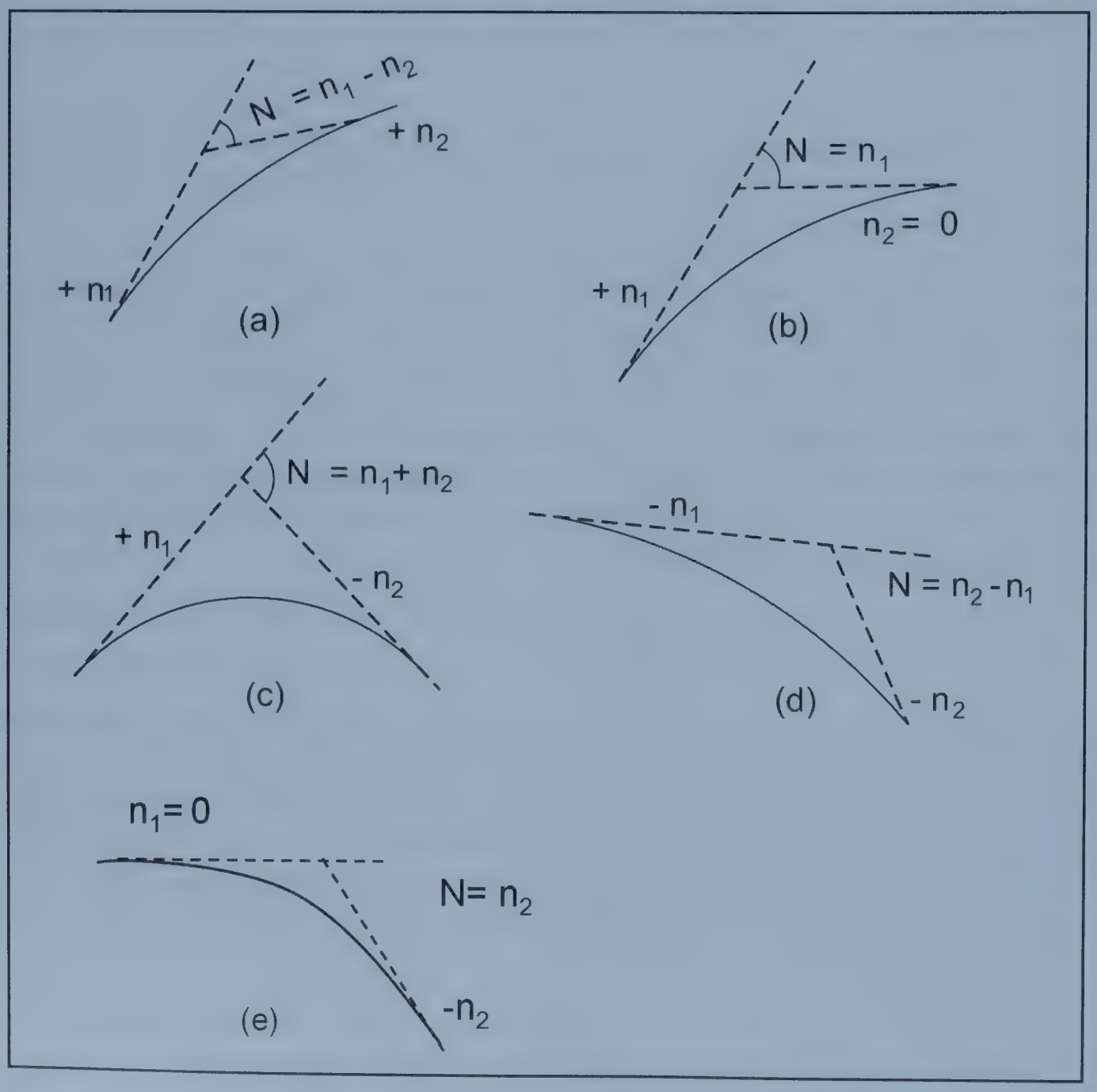


Fig. 7.2 Different Combinations of Grades resulting in a Summit Curve

The dynamics of movement over an ordinary summit curve is of little consequence. This can be inferred from two considerations:

a. The centrifugal force generated by the movement of the vehicle along the curve acts practically in opposition to the force of gravity and is, therefore, beneficial in so far as it relieves the pressure on the tyres and springs of the vehicle.

b. Vertical deviation angles on roads are generally small because the length of summit curves calculated based on sight distance criteria are long enough that shock is automatically rendered imperceptible to the vehicles.

It, therefore, follows that, on summit curves, transitions are not essential and simple circular arcs are good enough. Since a circular arc has a constant radius of curvature throughout its length, it gives a constant sight distance all along. In actual practice a simple parabolic curve is used instead of the circular arc. The reasons are:

a. A simple parabola is nearly congruent with a circular arc between the same tangent points, because of small vertical deviation angles and long lengths of curves.

b. A parabola is very easy to compute ordinates. For example, the equation of a parabola may be considered as:

$$y = \frac{x^2}{a} \qquad \ldots (7.3)$$

Considering that the property of the parabolic curve follows the details as shown in **Fig. 7.3**, the ordinate becomes,

$$y = \frac{Nx^2}{2L} \qquad \ldots (7.4)$$

where,

N = Angle of deviation, and

L = Length of curve ABC (= AC = AM) (since grade angles are very small)

S = Sight distance

And radius of the curve may be obtained as:

$$R = \frac{L}{N} \qquad \ldots (7.5)$$

7.5.2 *Length of a Summit Curve*

Length of a summit curve depends upon – (i) deviation angle, and (ii) sight distance to be considered. Sight distance considered can be 'minimum safe stopping sight distance' (SSD) or 'intermediate or overtaking sight distance' (i.e. ISD or OSD). There can be two cases which may appear with respect to the sight distance:

i. Length of the curve is more than the sight distance (**Fig. 7.3**), and

ii. Length of the curve is less than the sight distance (**Fig. 7.4**)

Considering the above conditions, the length of the summit curve can be calculated as follows:

a. Safe Stopping Sight Distance (SSD)

i. When length of the curve is more than the SSD:

$$L = \frac{NS^2}{(\sqrt{2H} + \sqrt{2h})^2} \quad \ldots (7.6)$$

where,

L = Length of summit curve in metres

S = Stopping sight distance, SSD, m

N = Deviation angle, equal to algebraic difference in grades, radians or tangent of the deviation angle

H = height of eye level of driver above road surface, m

h = height of subject above the pavement surface, m

The value of H, the driver's eye above roadway surface is taken as 1.2 m and the height of object 'h' above the pavement surface for the purpose of safe stopping sight distance is taken as 0.15 m as shown in **Fig. 7.5**. Substituting these values in the above **equation 7.6**, the length of summit curve is obtained as:

$$L = \frac{NS^2}{4.4} \quad \ldots (7.7)$$

ii. When length of the curve is less than the SSD:

$$L = 2S - \frac{(\sqrt{2H} + \sqrt{2h})^2}{N} \quad \ldots (7.8)$$

Here the description for L, S, N, H and h are the same as in **equation 7.6**. By substituting the values of H = 1.2 m and h = 0.15 m, the length of summit curve is obtained as:

$$L = 2S - \frac{4.4}{N} \quad \ldots (7.9)$$

b. Overtaking or Intermediate Sight Distance

i. When length of the curve is more than the OSD/ISD

The same general equation 7.6 is applicable in this case also. But, in this case, the values of H and h both are taken equal to 1.2 m. Substituting h = H in **equation 7.6** and simplifying:

$$L = \frac{NS^2}{8H} \quad \ldots (7.10)$$

In this case, the height of the eye level of driver as well as the height of the approaching object are taken as 1.2 m. Substituting the value of H, the height of eye level of driver above the pavement surface equal to 1.2 m:

$$L = \frac{NS^2}{9.6} \quad \ldots (7.11)$$

where,

L = Length of summit curve in meters

S = Overtaking or intermediate sight distance, (OSD/ISD), m

N = Deviation angle, equal to algebraic difference in grades, radians or tangent of the deviation angle

ii. When length of the curve is less than the OSD/ISD

The same general equation 7.8 may be used. By substituting h = H in equation 7.8 and simplifying:

$$L = 2S - \frac{8H}{N} \qquad \ldots (7.12)$$

Here again substituting the value of H as 1.2 m, the equation reduces to:

$$L = 2S - \frac{9.6}{N} \qquad \ldots (7.13)$$

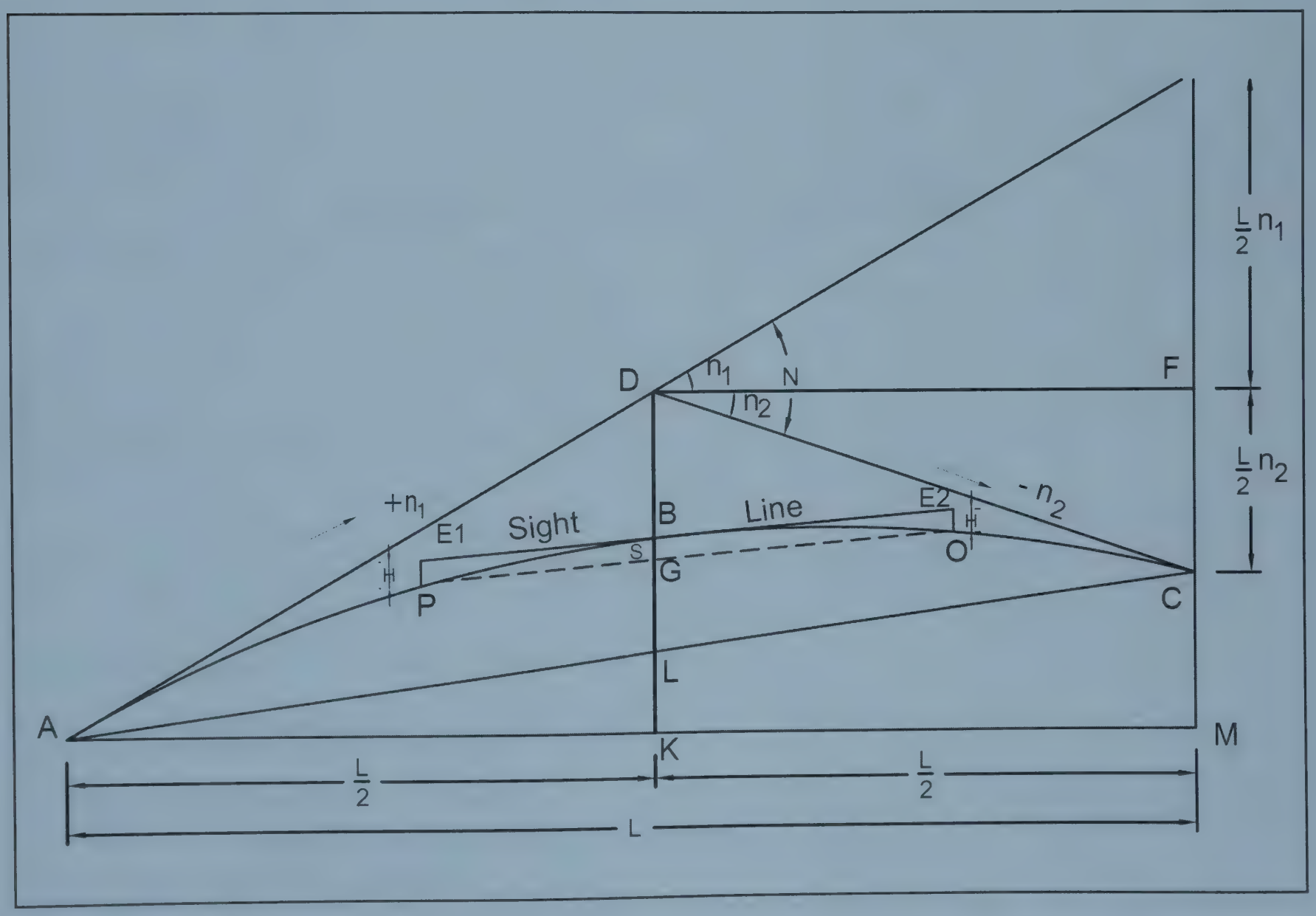


Fig. 7.3 Length of Summit Curve for L>S Case

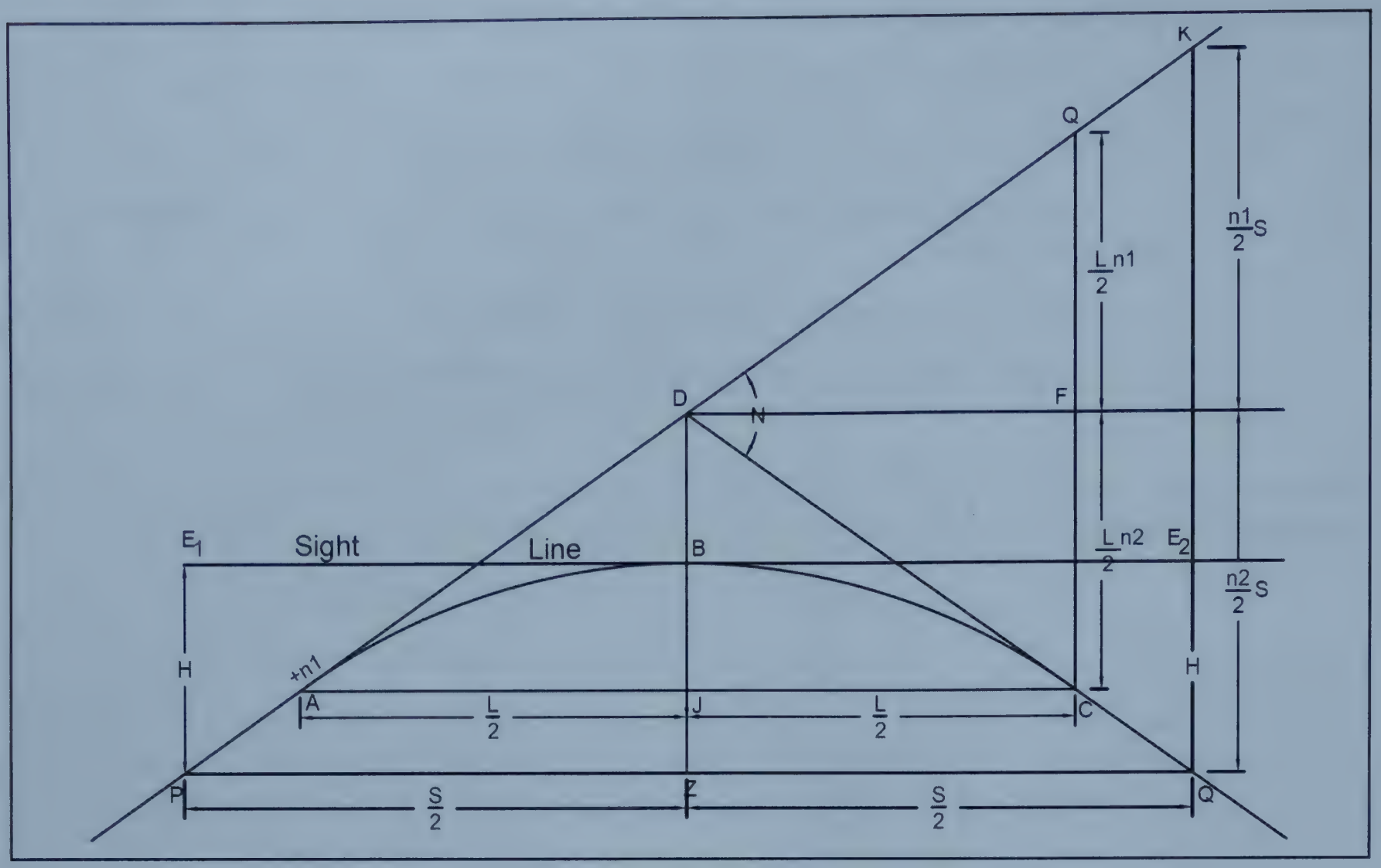


Fig. 7.4 Length of Summit Curve for L<S Case

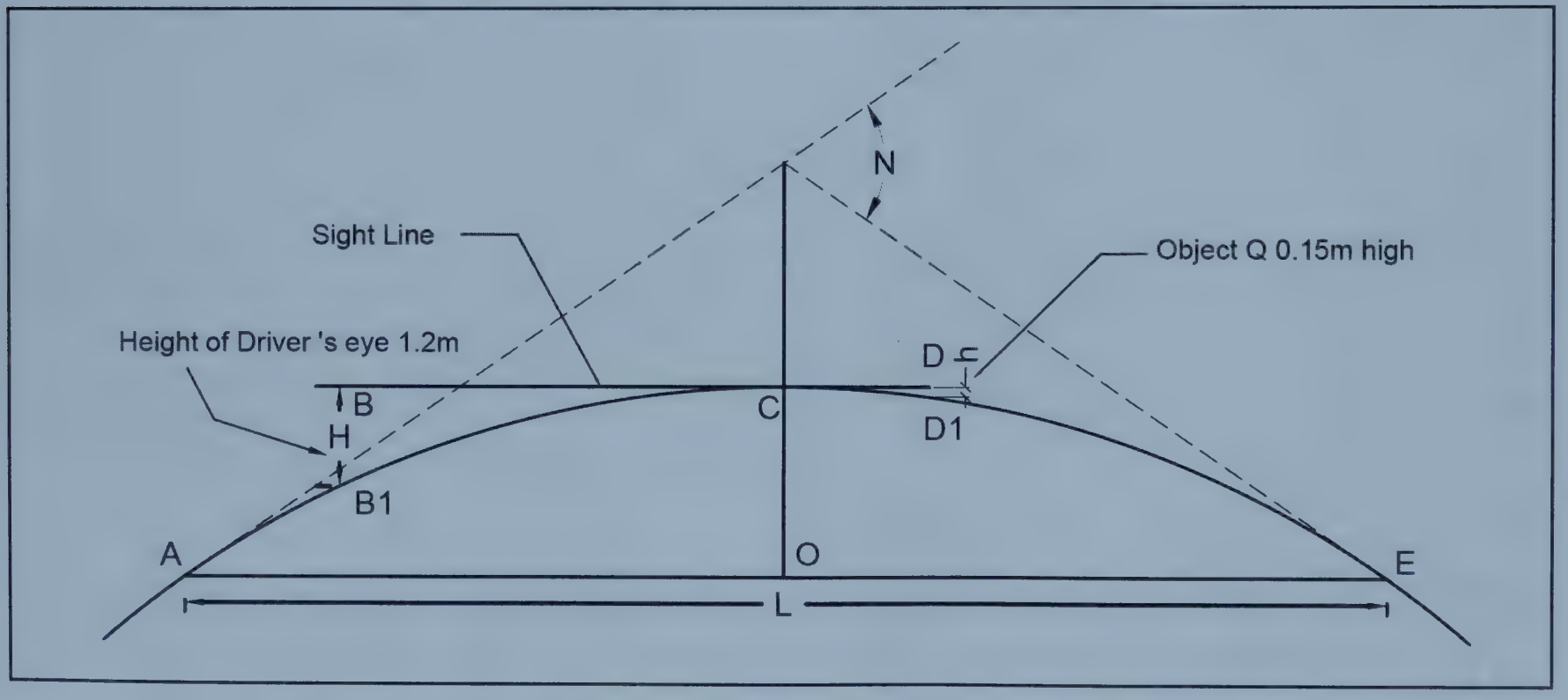


Fig. 7.5 Length of Summit Curve Considering SSD Criteria

The length of summit curve for various cases mentioned above can be read from **Figs. 7.18, 7.19** and **7.20**. In these figures, the value of ordinate 'M' to the curve from intersection point of the grade lines is shown.

For quick comparison of the length of summit curve for the three types of sight distances, for the cases when L is greater than S and when L less is than S, the lengths are shown in terms of the grade difference in **Table 7.3** and **Table 7.4** respectively.

Table 7.3 Length of Vertical Curve for different Speeds (L>S)

Design Speed (km/h)	Length of Summit Curve (m) for			Length of Valley Curve for Headlight Sight Distance(m)
	Stopping Sight Distance	Intermediate Sight Distance	Overtaking Sight Distance	
20	0.9N	1.7N	-	1.8N
25	1.4N.	2.6N	-	2.6N
30	2.0N	3.8N	-	3.5N
35	3.6N	6.7N	-	5.5N
40	4.6N	8.4N	28.4N	6.6N.
50	8.2N	15.0N	57.5N	10.0N
60	14.5N	26.7N	93.7N	14.9N
65	18.4N	33.8N	120.4N	17.4N
80	38.4N	70.4N	230.1N	27.9N
100	73.6N	135.0N	426.7N	41.5N

Note: *Deviation angle 'N' is expressed as percentage*

Table 7.4 Length of Vertical Curve for different Speeds (L<S)

Design Speed (km/h)	Length of Summit Curve (m) for			Length of Valley Curve for Headlight Sight Distance(m)
	Stopping Sight Distance	Intermediate Sight Distance	Overtaking Sight Distance	
20	40 - 4.4/N	80 - 9.6/N	-	40 - 2.2/N
25	50 - 4.4/N	100 - 9.6/N	-	50 - 2.4/N
30	60 - 4.4/N	120 - 9.6/N	-	60 - 2.6/N
35	80 - 4.4/N	160 - 9.6/N	-	80 - 2.9/N
40	90 - 4.4/N	180 - 9.6/N	330 - 9.6/N	90 - 3.1/N
50	120 - 4.4/N	240 - 9.6/N	470 - 9.6/N	120 - 3.6/N
60	160 - 4.4/N	320 - 9.6/N	600 - 9.6/N	160 - 4.3/N
65	180 - 4.4/N	360 - 9.6/N	680 - 9.6/N	180 - 4.7/N
80	260 - 4.4/N	520 - 9.6/N	940 - 9.6/N	260 - 6.1/N
100	360 - 4.4/N	720 - 9.6/N	1280 - 9.6/N	360 - 7.8/N

Note: *Deviation angle 'N' is expressed as percentage*

7.5.3 *Minimum Length of Vertical Curve*

It is seen that the length L of a summit curve decreases as N and/or S decreases. Therefore, in some cases the length of the curve needed for providing the required sight distance would be very small. Further in flat grades no vertical curve may be necessary for visibility; but for comfort in driving and to avoid shock, it is necessary to introduce a summit curve except in case of very flat grades. The minimum length of the curve should be as indicated in **Table 7.5.** This table also shows the maximum grade change not requiring a vertical curve.

Table 7.5 Minimum Length of the Vertical Curve

Design Speed (km/h)	Maximum Grade Change (percent) up to which Vertical Curve is not required	Minimum Length of a Vertical Curve (metre)
Up to 35	1.5	15
40	1.2	20
50	1.0	30
60	0.8	40
80	0.6	50 / 70*
100	0.5	60 / 85*
120	0.5	100*

Note: *Minimum length of vertical curves shown in asterisk (*) are applicable for expressways.*

7.5.4 *Highest Point on Summit Curve*

It is sometimes important to know the position of the highest point on a vertical curve for the purpose of layout of drainage appurtenances and for ascertaining vertical clearances in restricted locations i.e. road under bridges, etc. When the two grades are equal, the curve would be symmetrical about the vertical bisector of the intersecting angle and the highest point would also lie on this bisector. When the two grades are unequal the curve would be tilted and the highest point of the curve would lie on the side of the flatter gradient. The distance of highest point from the start of the curve is given by **equation 7.14** and represented in **Fig. 7.6**.

$$X_0 = \left(\frac{n_1}{n_1+n_2}\right) L \qquad \ldots (7.14)$$

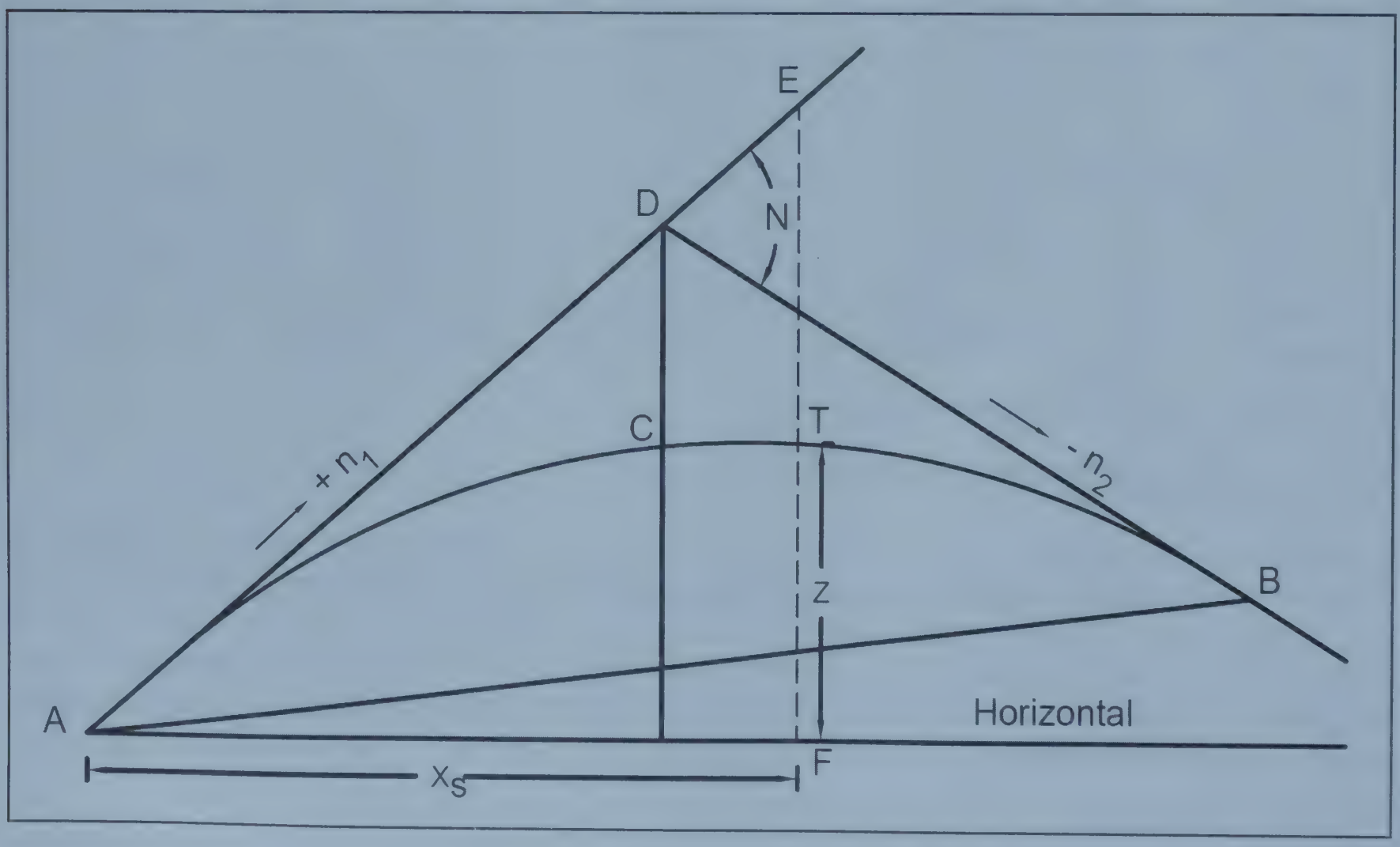


Fig. 7.6 Highest Point on Summit Curve

Ordinate from tangent is: $y = \frac{x^2}{a}$, where 'x' is the length of the chord considered (say 30 m) Second ordinate can be calculated as: $y_2 = y_1 * 2^2$ or in general as $y_1 * r^2$.

Ordinate of summit curve from the point of intersection of the grade lines:

$$M = \frac{NL}{8} \qquad \ldots (7.15)$$

Considering the above and satisfying the minimum length of ordinate possible, the minimum length of the curve is as shown in **Table 7.6**.

Table 7.6 Minimum Length of Summit Curve Satisfying Ordinate Requirement for SSD

Speed, km/h	L < S, Minimum Ordinate M = 0.3		L > S, Minimum Ordinate M = 0.6	
	Length of Curve (m)	Deviation Angle (N)	Length of Curve (m)	Deviation Angle (N)
30	20	0.11	30	0.16
40	35	0.075	50	0.0105
50	40	0.056	65	0.075
65	70	0.037	95	0.05
80	90	0.03	130	0.037
100	130	0.018	195	0.025

7.5.5 *Design of Summit Curves*

The design of summit curves follows the procedure given below:

a. On a longitudinal section of the road, drawn to scale, the economical gradients are fixed and selected by taking into consideration the amount of earthwork and other incidental works involved. The value of two gradients meeting at a point being known, the deviation angle N is known.

b. The sight distance applicable to the section of the road is selected, considering the classification of the road, the topography of the area and whether the section lies in an overtaking zone or non-overtaking zone.

c. The value of N and S being thus known, the appropriate length of the summit curve, L, corresponding to these values can be taken from **Figs. 7.18, 7.19** or **7.20**, as applicable.

d. The value of L as read from the graph is then rounded off so that the modified value is divisible into a number of equal chords of a reasonable length not exceeding R/200, where R is the radius of the curve at the apex given by R = L/N.

e. By reading the value of M for the length designed from the graph, the depth of cutting required for constructing the curve is obtained. This depth may be checked to see if the cutting would be excessive.

f. The constant "a" is calculated and the ordinate $y_r = a * r^2$ is obtained. The other ordinates and reduced levels of the various station points on the curve are then calculated and tabulated to facilitate setting out in the field.

7.6 Valley Curves

7.6.1 *General*

A vertical curve concave upwards is known as a valley curve. Combination of different grade lines (upward or downward) will result in different orientations of the valley curve. These are shown in **Fig. 7.7**.

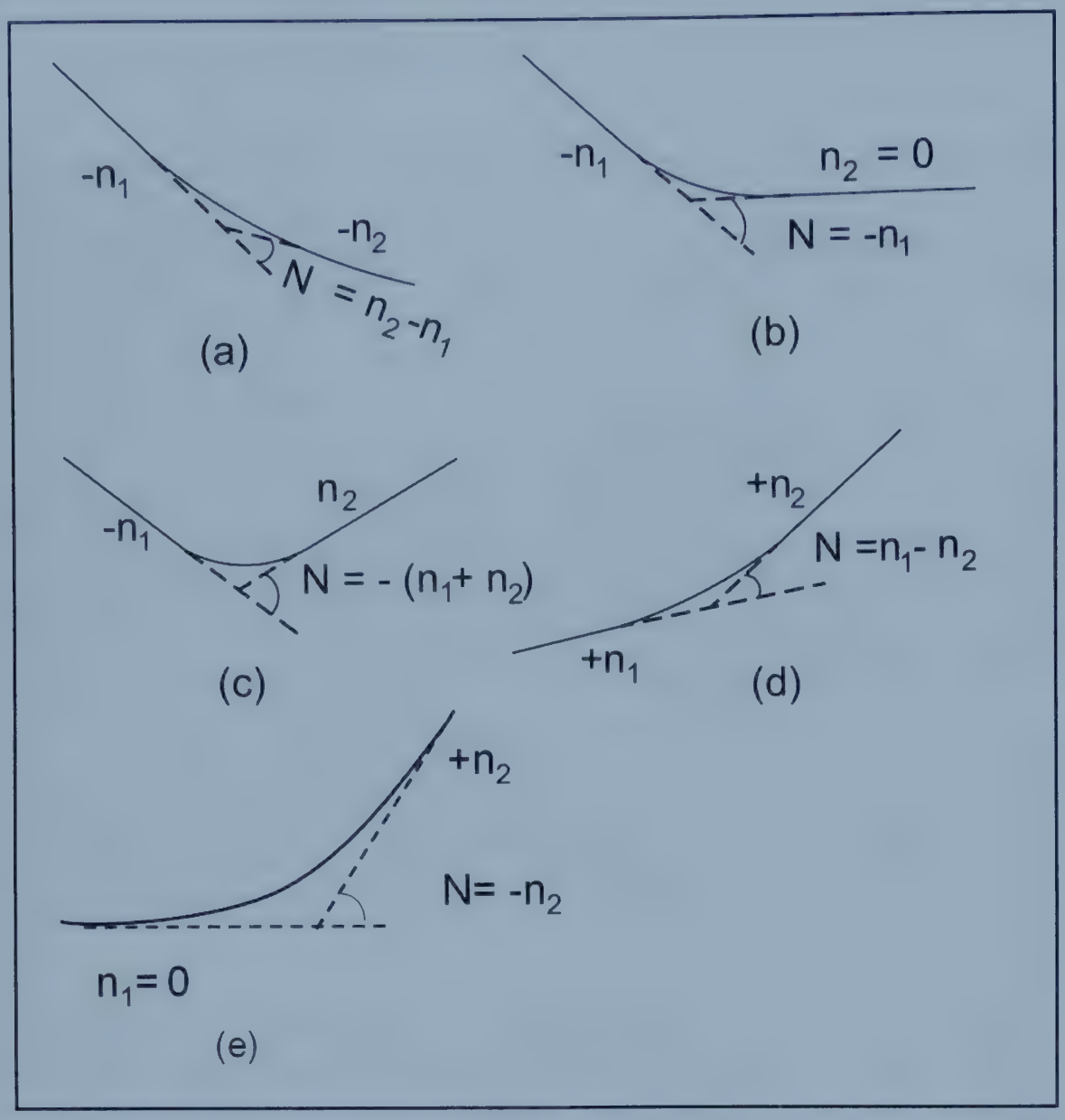


Fig. 7.7 Types of Configurations of Valley Curve Based on Grade Lines

Valley curves should have the shape of square parabola similar to summit curves.

7.6.2 *Length of Valley Curves*

A number of criteria are available for establishing the lengths of valley curves. Most commonly used among these are:

(i) Headlight sight distance

(ii) Rider comfort

When a vehicle traverses a valley curve at night, the portion of road lighted ahead depends on the height of the headlights above the road surface and the direction of the light beam. The valley curve should be long enough so that the distance ahead lighted by the headlights is at least equal to the safe stopping sight distance as shown in **Fig. 7.8.** On valley curves, the gravitational and centrifugal forces act together resulting in extra pressure on the tyres and

springs of the vehicle. The effect of this on driving comfort depends on several factors such as the vehicle body suspension, tyre flexibility, weight carried, etc. The broad conclusions from limited observations show that for riding comfort on valley curves, the radial acceleration should not exceed 0.3 meter per second per second. The length of vertical curve required to satisfy this comfort factor is only about 75 per cent of that required to satisfy the headlight sight distance requirement. It is, therefore, recommended that the length of valley curves for design should be based on the considerations of headlight sight distance. The values as derived should, however, be subjected to the minimum lengths indicated in **Table 7.4**.

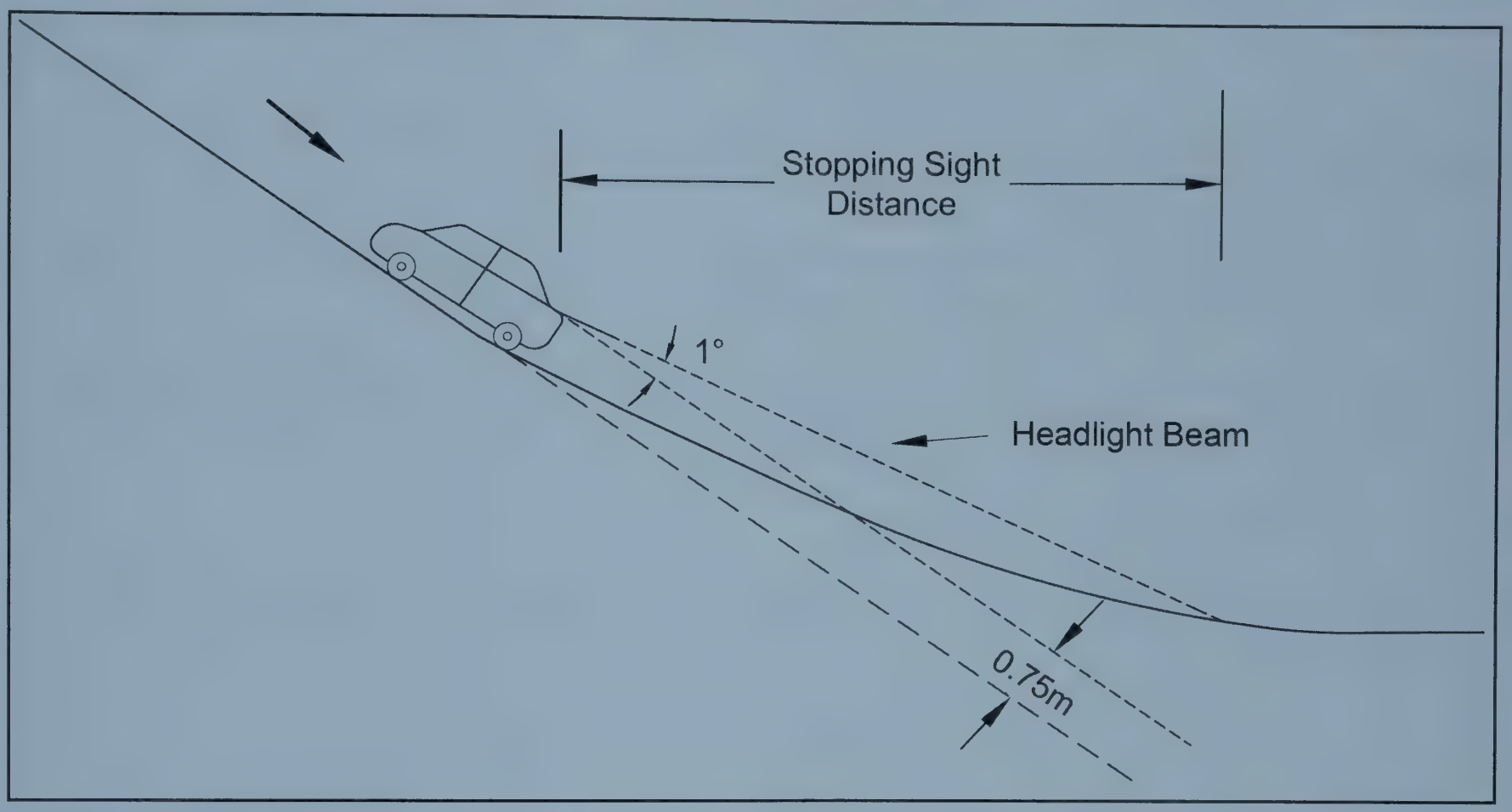


Fig. 7.8 Length of Valley Curve as per Head Light Sight Distance Criteria

For determining the length of valley curves based on the above considerations the following criteria apply:

(i) Height of headlight above road surface is 0.75 m

(ii) The useful beam of headlight is up to one degree upwards from the grade of the road,

(iii) The height of object is nil

Similar to summit curve, the length can be calculated for two cases: Length of curve exceeds the required sight distance (L > S)

$$L = \frac{NS^2}{(2h_1 + 2S\tan\propto)} \qquad \ldots (7.16)$$

Applying the above criteria, average height of the head light, h_1 is taken as 0.75 m and the beam angle, $\propto$ = 1°, substituting these in the above equation 7.16,

$$L = \frac{NS^2}{1.50 + 0.035S} \qquad \ldots (7.17)$$

where, L = total length of valley curve, m

S = SSD, m

N = deviation angle = $(n_1 + n_2)$, with slopes $-n_1$ and $+n_2$

Length of curve is less than the required sight distance (L < S)

$$L = 2S - \frac{(2h_1 + 2S\tan\alpha)}{N} \quad \ldots (7.18)$$

Substituting h_1 = 0.75 m and = 1°,

$$L = 2S - \frac{1.50 + 0.035\,S}{N} \quad \ldots (7.19)$$

Note: *While putting the value of deviation angle (N) in the above equations (for calculating length of the valley curves), absolute value of N has to be considered.*

Length of valley curve for the above two cases can also be found out using **Tables 7.3** and **7.4** and minimum length of the curve can be found out using **Table 7.5**.

Drainage considerations become important, for valley curves between a descending grade and an ascending grade. For drainage purposes, it is desirable that the curve has a minimum gradient as per the provisions of IRC:SP:42 and IRC:SP:50.

7.6.3 *Lowest Point on Valley Curve*

From the geometry of the curve shown in **Fig. 7.9**, the lowest point on the curve from the start (i.e. A) will be at a distance:

$$X_0 = \left(\frac{n_1}{n_1 + n_2}\right) L \quad \ldots (7.20)$$

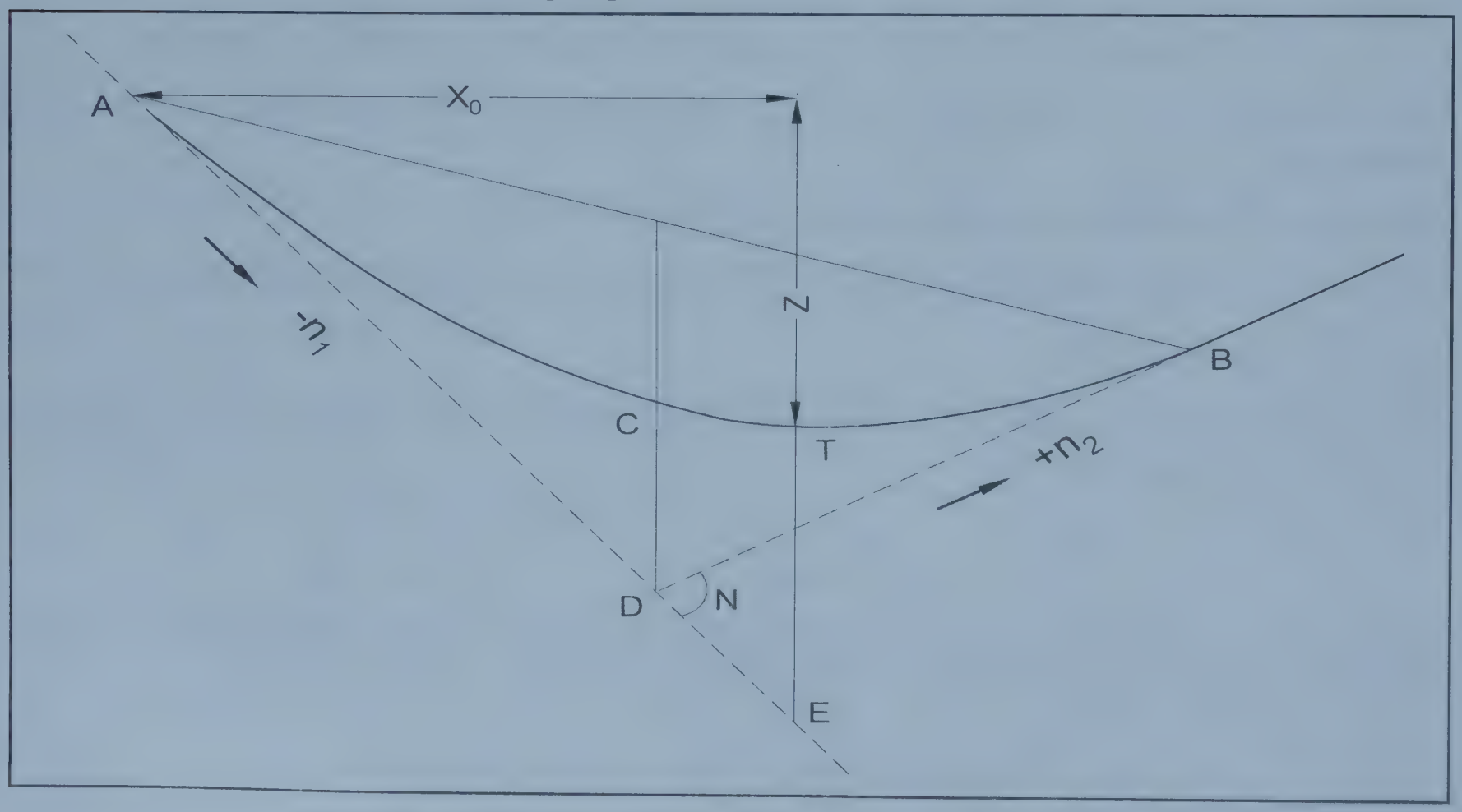
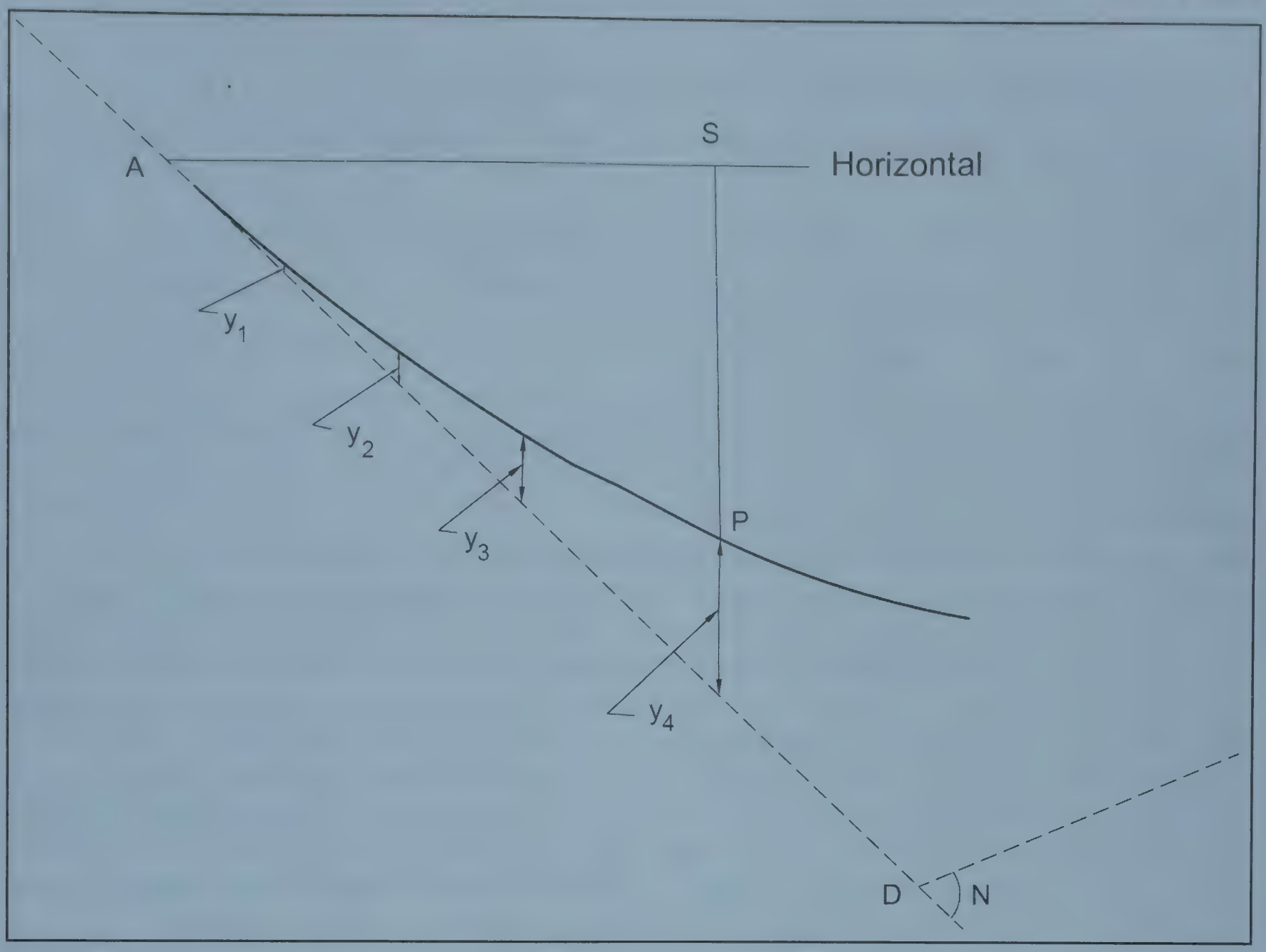


Fig. 7.9 Lowest Point on a Valley Curve



From **Fig. 7.10**, the ordinates can be worked out.

Fig. 7.10 Computation of Ordinates

7.6.4 *Design of Valley Curves*

The design of valley curves follows the procedure given below:

(i) The gradient lines are marked on the longitudinal section of the road and the deviation angle, N, calculated.

(ii) The design speed, V, appropriate to the class of road and the topography of the area, is decided.

(iii) By using the **Fig. 7.21**, the length of the curve for the corresponding values of N and V is obtained. This length, L, is rounded off so as to be divisible into a number of equal chords of a convenient length not exceeding R/200.

(iv) The constant "a" and the ordinate “y”are obtained and reduced levels of the station points are calculated from these and tabulated for setting out in the field

7.6.5 *Measurement of Vertical Sight Distance*

The longitudinal sections are plotted to the following scales:

(i) Built-up areas and stretches in hilly terrain 1:1000 for horizontal scale and 1:100 for vertical scale.

(ii) Plain and rolling terrain 1:2500 for horizontal scale and 1:250 for vertical scale.

If L is the length of the vertical curve required for gradient N, its radius R is equal to L/N. The radius, r, for purpose of plotting is then $\frac{RV}{H^2}$,where V is the vertical scale of the drawing and H is the horizontal scale of the drawing. Having known the value of 'r', the vertical curve is easily drawn on the profile with the aid of spline or railway curves. For the recommended scales of plotting having vertical to horizontal scale ratio of 1:10, the error in measurement of sight distance will not be more than about 5 per cent.

Measurement of vertical sight distance at summit curves may be done from plotted profiles of the highway by the method illustrated in **Fig. 7.11.** A transparent straight edge with parallel edges 1.2 m apart and a dotted line 0.15 m from the upper edge, as per the vertical scale of the profile, is the tool employed for these measurements. The transparent strip is placed on the profile with the lower edge at the station for which the available sight distance is desired and the strip revolved about this point until the upper edge touches the profile. Stopping sight distance available is then the distance between the first station and the point of intersection of the 0.15 m line with the profile. Overtaking/intermediate sight distance, in similar manner, is the distance between the initial station and the point where lower edge of the strip meets the profile. If overhead obstructions to visibility like under-bridges, etc. have also been marked on the profile, then the graphical method explained above will unveil visibility deficiencies caused by these.

Availability of headlight sight distance along valley curves can also be checked in a similar way except that the template for checking will be different as explained in **Fig. 7.12.** At the point where the sight distance is to be checked, the lower edge of the template is kept tangential to the road surface, and the headlight distance is the distance between starting station and the point of intersection between the road profile and light ray line (marked 1° upward from the headlight).

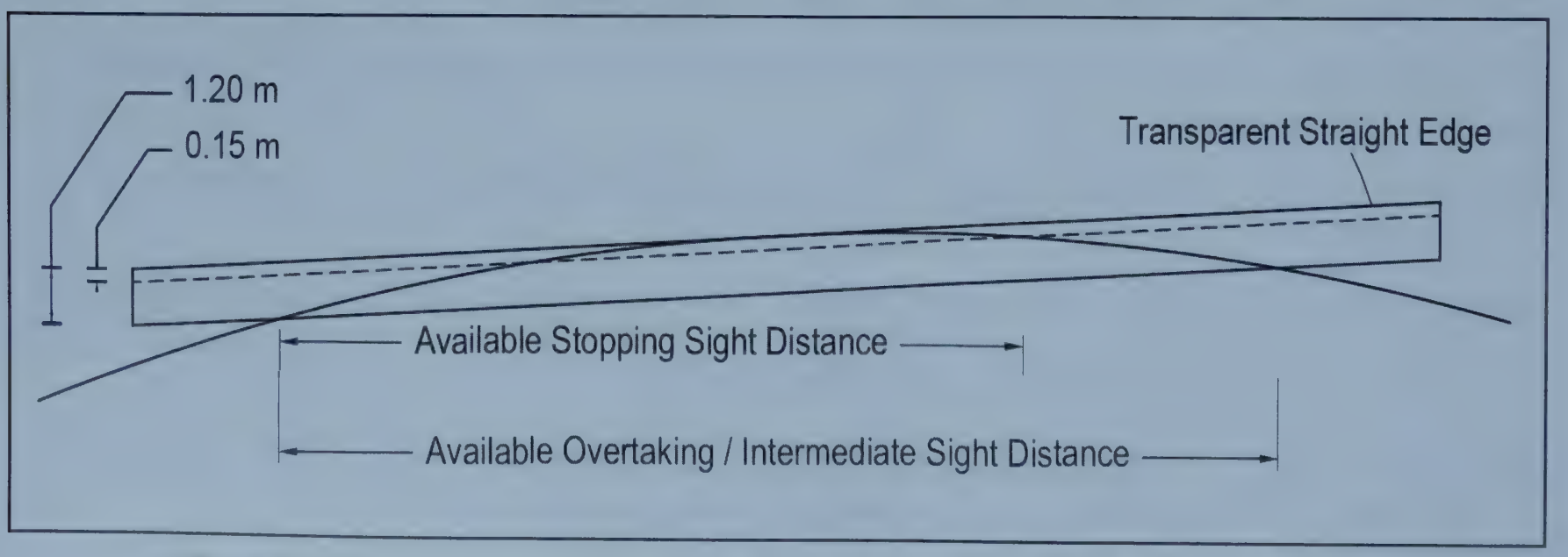


Fig. 7.11 Measurement of Vertical Sight Distance on a Summit Curve

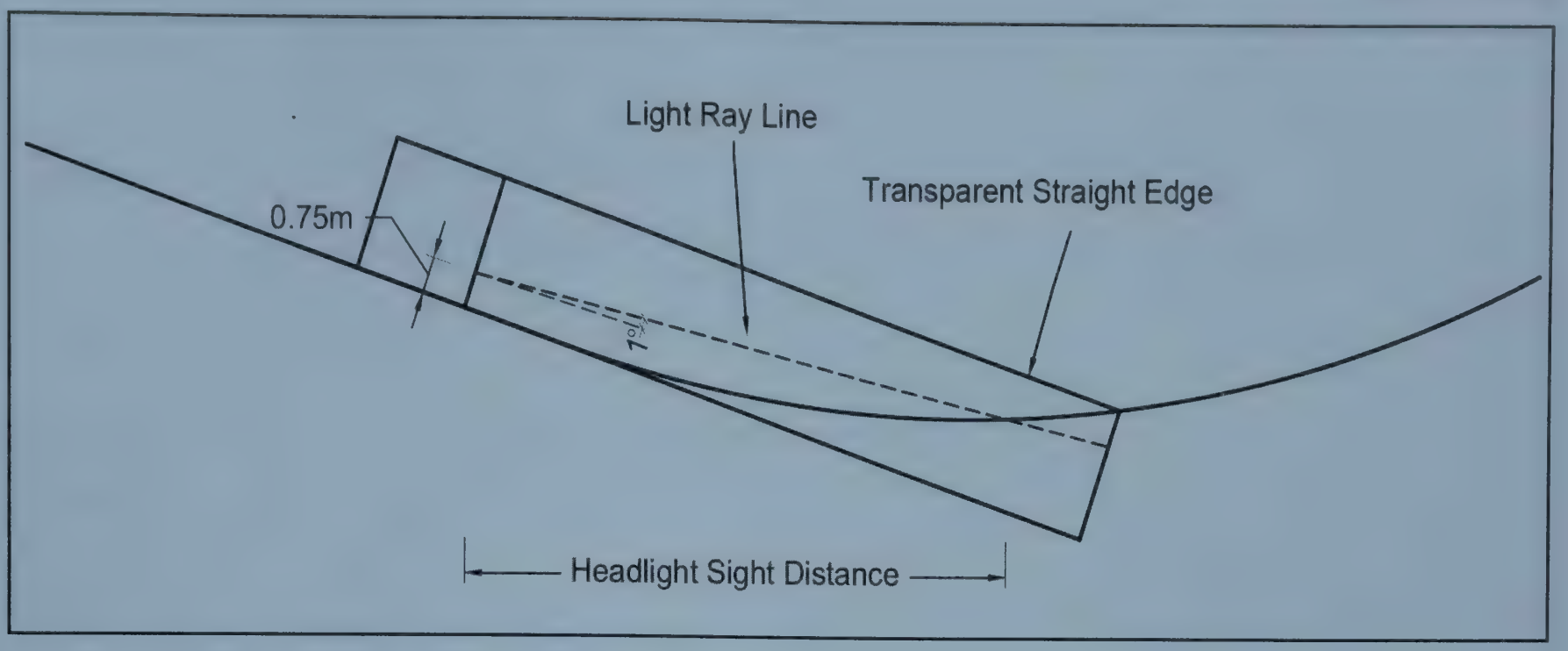


Fig. 7.12 Measurement of Vertical Sight Distance on a Valley Curve

7.7 Illustrative Examples

Problem 1: Calculate the length of the vertical curve for the given data (**Fig. 7.13**):

Gradients: n_1 = +1/25, n_2 = -1/30

Road: National Highway in plain terrain

Solution: Deviation angle N = n_1 – (n_2) = 0.073

Design speed for NH in plain terrain = 100 km/h

Sight Distance: SSD = 180 m, and OSD = 640 m (From **Tables 5.2** and **5.3**)

Length of vertical Curve: Curve is summit in nature

a) For SSD 180 m and deviation angle 0.073 the length of curve is 540 m from **Fig. 7.18**.
Radius of curvature = L/N = 7397.3 m

Constant 'a' for parabola = 2L/N = 14795

Calculation of ordinates $y_1 = \frac{u^2}{a}$, where 'u' is length of chord. This may be taken as 30 m by dividing length of curve in 18 equal parts,

First ordinate y_1= (30*30/14795) = 0.061

Highest point on curve, $X_0 = \left(\frac{n_1}{n_1 + n_2}\right) L$ = 295.89 m

Reduced level of this point: Say RL of starting point of curve is 100.000 m.

RL of point of intersection = 100 + 295.89*(1/25) = 111.84 m

Ordinate from point of intersection: $Y_{max} = \left(\frac{295.89}{30}\right)^2 x\, y_1$ = 5.915 m

RL of highest point = 111.84 – 5.915 = 105.925 m

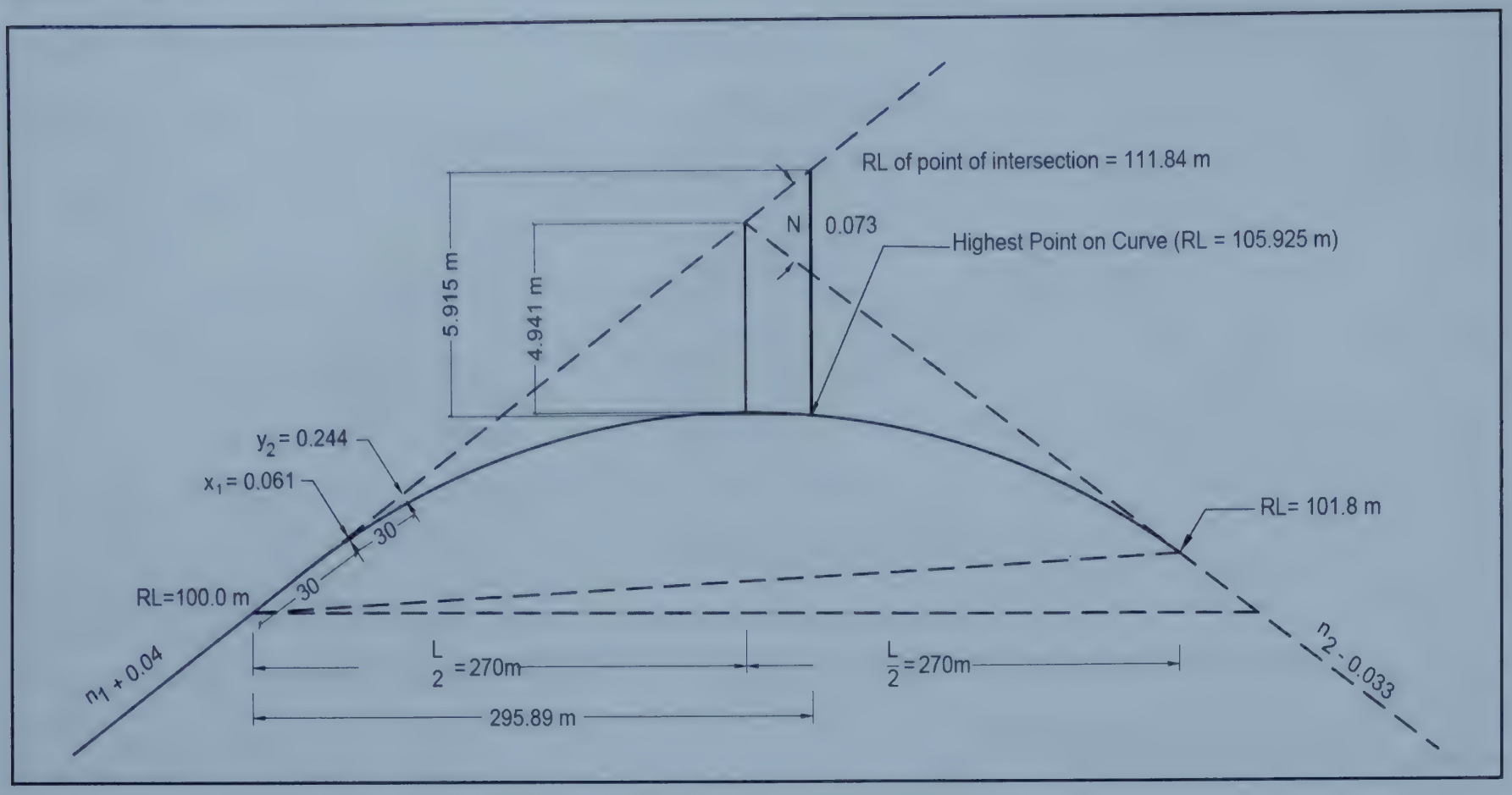


Fig. 7.13 Sketch of Problem 1

b) For OSD 640 m

From **Fig. 7.20** corresponding to N = 0. 073 and V= 100 km/h the intersection point giving length of the curve for the given conditions is outside the charts. Therefore, using equation:

$$L = \frac{NS^2}{9.6}$$

L = 3115 m, round of to 3200 m.

The radius of curvature of the curve is 43,836 m. Divide the curve into 32 stations of 100 m each.

Design calculations:

Constant: a = $\frac{2L}{N}$ = 87671 m

Ordinate from tangent line = $y_1 = \frac{u^2}{a}$ = (100 x 100)/87671 = 0.114 m

Highest point of the curve occurs at x_o

$$X_0 = \left(\frac{n_1}{n_1 + n_2}\right) L = 1753.4$$

Ordinate from point of intersection y_{max}= 17.53^2 x 0.114 = 35.03 m

R.L. along the 4 per cent grade = 100.00+17.53 x 4 = 170.12 m

R. L. of the highest point = 170.12 - 35.03 = 135.09

The design values are shown in the **Fig. 7.14**. Design values corresponding to problem 1 are mentioned in **Table 7.7** (for SSD) and **Table 7.8** (for OSD) respectively.

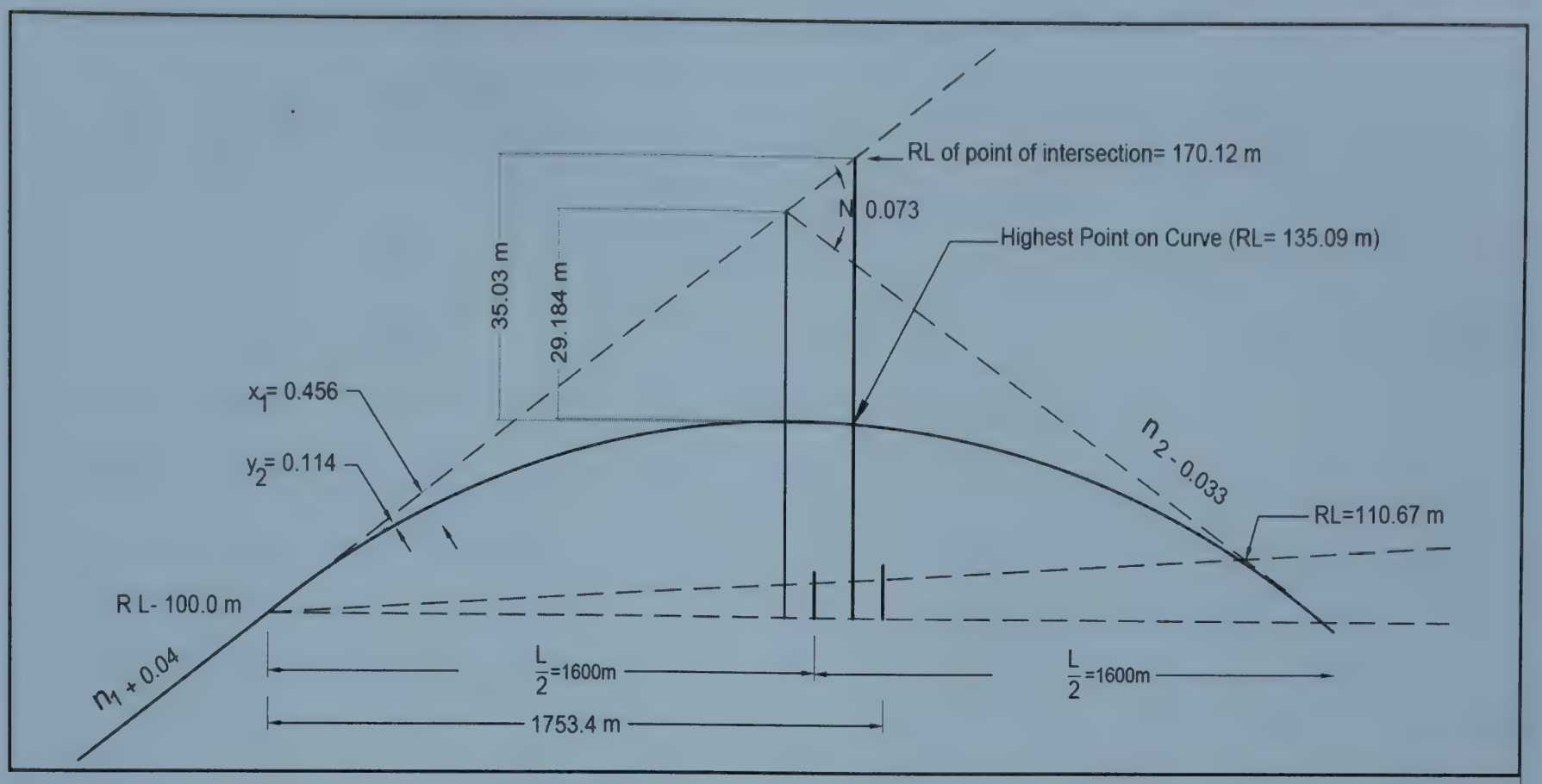


Fig. 7.14 Design Values of Problem 1 for OSD

Table 7.7 Design Table for Problem 1 Case SSD

First half of curve (1 in 25)					Second half of curve (1 in 30)				
No. of station from B.V.C.	Chainage from B.V.C.	R.L. of points on grade line	Ordinates between curve and grade line	R.L. of station on curve	No. of station from B.V.C.	Chainage from B.V.C.	R.L. of points on grade line	Ordinates between curve and grade line**	R.L. of station on curve
Read downwards									
0 (B.V.C.)	0	100.000	0.000	100.00	18 (E.V.C.)	540	101.800	0.000	101.800
1	30	101.200	y_1= 0.061	101.139	17	510	102.800	0.061	102.739
2	60	102.400	2^2y_1=0.244	102.156	16	480	103.800	0.244	103.556
3	90	103.600	3^2y_1=0.549	103.051	15	450	104.800	0.549	104.241
4	120	104.800	4^2y_1=0.976	103.824	14	420	105.800	0.976	104.824
5	150	106.000	5^2y_1=1.525	104.475	13	390	106.800	1.525	105.275
6	180	107.200	6^2y_1=2.196	105 004	12	360	107.800	2.196	105.604
7	210	108.400	7^2y_1=2.989	105.411	11	330	108.800	2.989	105.811
8	240	109.600	8^2y_1=3.904	105.796	10	300	109.800	3.904	105.896
9	270	110.800	9^2y_1=4.941	105.859	9	270	110.800	4.941	105.859
					Read upwards				

Notes: *B.V.C.* = Beginning of the vertical curve;

E.V.C. = End of the vertical curve;

**Same values as in opposite station of first half of the curve

Table 7.8 Design Table for Problem 1 Case OSD

First half of curve (1 in 25)					Second half of curve (1 in 30)				
No. of station from B.V.C.	Chainage from B.V.C.	R.L. of points on grade line	Ordinates between curve and grade line	R.L. of station on curve	No. of station from B.V.C.	Chainage from B.V.C.	R.L. of points on grade line	Ordinates between curve and grade line**	R.L. of station on curve
Read downwards									
0 (B.V.C.)	0	100.000	0.000	100.00	32	3200	110.667	0.000	110.667
1	100	104.000	y_1= 0.114	103.886	31	3100	114.000	0.114	113.886
2	200	108.000	2^2 y_1=0.456	107.544	30	3000	117.334	0.456	116.878
3	300	112.000	3^2 y_1=1.026	110.974	29	2900	120.667	1.026	119.641
4	400	116.000	4^2 y_1=1.824	114.176	28	2800	124.000	1.824	122.176
5	500	120.000	5^2 y_1=2.850	117.150	27	2700	127.334	2.850	124.484
6	600	124.000	6^2 y_1=4.104	119.896	26	2600	130.667	4.104	126.563
11	1100	144.000	11^2y_1=13.794	130.206	21	2100	147.334	13.794	133.540
12	1200	148.000	12^2y_1=16.416	131.584	20	2000	150.667	16.416	134.251
13	1300	152.000	13^2y_1=19.266	132.734	19	1900	154.000	19.266	134.734
14	1400	156.000	14^2Y_1=22.344	133.656	18	1800	157.334	22.344	134.990
15	1500	160.000	15^2Y_1=25.650	134.350	17	1700	160.667	25.650	135.017
16	1600	164.000	16^2Y_1=29.184	134.816	16	1600	164.000	29.184	134.816
					Read upwards				

**Same values as in opposite station of first half of the curve

Problem 2: For the case in problem 1, site conditions do not permit the provision of a summit curve exceeding 300 m in length. For stopping sight distance considerations, find:

(i) The adjustments required in the grades for permitting the design speed of 100 km/h, and

(ii) The safe speed if the grades are not changed

Solution: Adjustment of the approach gradients

For the speed of 100 km/h, S = 180 m (**Table 5.2**), L as given = 300 m

$$N = \frac{4.4L}{S^2} = 0.040$$

The gradients should be so adjusted that their algebraic difference is 0.04.

(ii) Limiting the safe speed

$$S = \left(\frac{4.4L}{N}\right)^{0.5} = 134.5$$

The safe speed corresponding to stopping sight distance of 134.5 m = 85 km/h.

Problem 3: Design the approach to a long bridge on a National Highway in plain terrain. The deck of the bridge is 5.5 m above the general road level. Provide for intermediate sight

distance. (**Fig. 7.15**)

Solution: Design speed = 100 km/h

S (intermediate) = 360 m

From **Fig. 7.15**, ignoring the straight-line grade CD, $\frac{1}{2}(L_s-(-L_v)) \times N = 5.5$ m

Try N = 0.025 (i.e. 1 in 40)

L_s (from **Fig. 7.19**) = 337.5 m or 340, L_v (from **Fig. 7.21**) = 70 m

Therefore, ½ (340 + 70) x 0.025 = 5.125 m

This is near enough and the balance fall (5.5 - 5.125) or 0.375 m will be covered by straight grade portion CD

Length of CD = 0.375/0.025=15 m

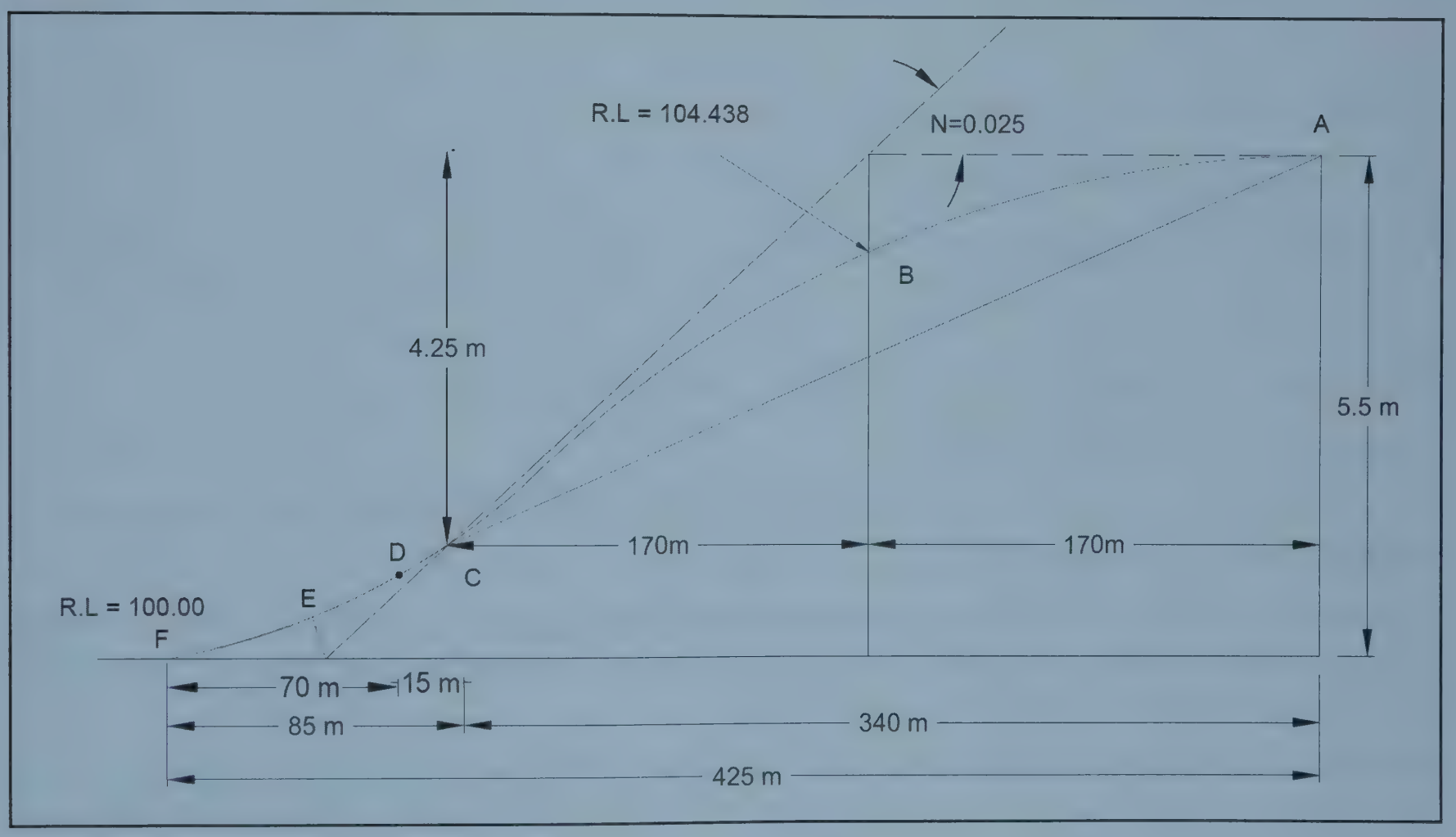


Fig. 7.15 Sketch of Problem 3

(i) Design of summit curve

Divide the length of the curve in chords 30 m each. The design chart may be calculated as under:

Constant 'a' = $\frac{2L}{N}$ = 2 x 340 /0. 025 = 27200

First ordinate y_1= 30 x 30 / 27200 = 0.033

The other ordinates can be calculated and the levels on the curve worked out similar to Problem 1.

(ii) Design of valley curve

Divide the length of the curve in chords 10 m each. The design chart may be prepared as under:

Constant 'a' = 5600

First ordinate y_1= 10 x 10 / 5600 = 0.0179

Since valley curve is also of square parabola, the other ordinates can be calculated similar to summit curves.

The total length of one side bridge approach works out to 425 m.

Problem 4: For the case in Problem 3, work out the profile for a gradient of 1 in 50 (2 per cent) and compare with the profile obtained with the gradient of 1 in 40 (Problem 3).

Solution: S = 360 m; N = 0.02; L_s (from **Fig. 7.19**) = 240 m; L_v (minimum) = 60 m

Assuming the general level of the road to be R.L. 100.0, referring to previous Figure,

R.L. of F = 100.0 m

- do - D = 100 + ½ x 60 x 0.02= 100.6 m

- do - E = 100 + 0.6 / 4 = 100.15 m

- do - C = 105.5 – ½ x 240 x 0.02 = 103.1 m

- do - B = 105.5 – 2.4/4 = 104.9 m

Difference in level between C and D = 103.1-100.6 = 2.5

Length of CD = 2.5 x 50 = 125 m

Total length of approach = 240+60+125 = 425 m which is the same as that obtained in problem 3 with 1 in 40 gradient.

Problem 5: An urban arterial having divided carriageway is to cross a railway line over a bridge 25 m span. The difference in deck level of the bridge and the general road level is 6.0 m. Design suitable profile for the approaches. (**Fig. 7.16**)

Solution: As the location is in urban area where a lot of slow-moving traffic is expected, it is preferable to adopt a flat gradient of 1 in 40 to 1 in 50. For the present case, adopt a gradient of 1 in 50 or 0.02.

Stopping sight distance will apply for divided carriageways. For the design speed of 80 km/h, S = 120 m

(i) Design of summit curve

As the bridge span is short, less than 30 m, it will be preferable to provide a single summit curve encompassing the bridge deck as well.

N = 0. 02 + 0. 02 = 0. 04

L (from **Fig. 7.18**) = 131 m, or adopt 150 m

R = L/N = 150/ 0.04 = 3750 m

The maximum difference in level between the horizontal and the curve at the centre of bridge (see Figure).

$= \frac{l^2}{8R}$ = (25 x 25)/ (8 x 3750) = 0.021 = 2.1 cm

This can be accommodated in the wearing course

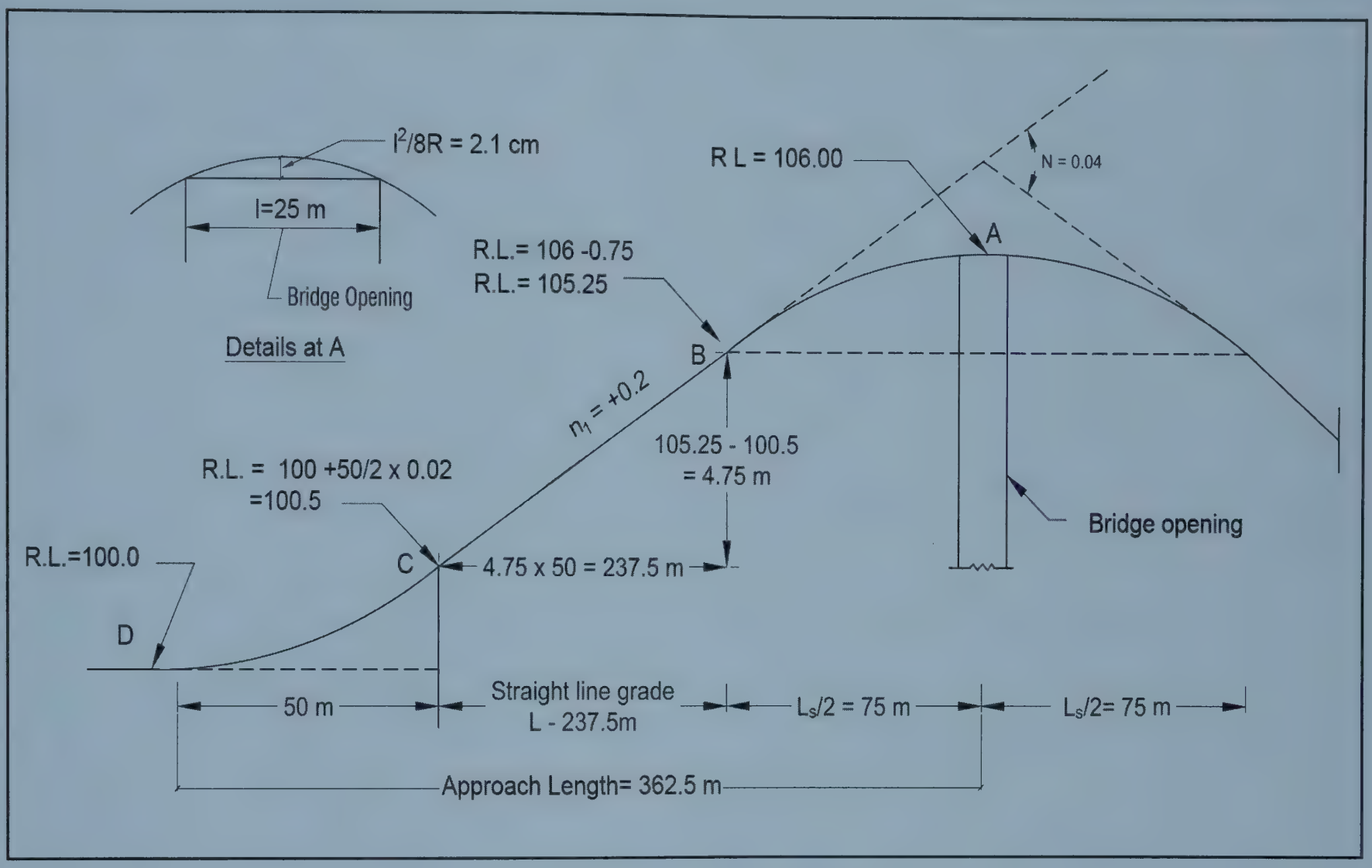


Fig. 7.16 Sketch of Problem 5

(ii) Valley curve

For the gradient of N = 0.02 and design speed of 80 km/h, the minimum lengths will apply. That is, the length of valley curve on either side will be 50 m.

The proposed profile is shown in **Fig. 7.16.**

Problem 6: The deck of a slab culvert on a National Highway is 0.6 m above the general road level on the flanks. Design suitable shock-free curves for the culvert approaches.

Solution: As the hump caused by the culvert will not obstruct the visibility, the minimum lengths from riding comfort considerations will govern the design. (**Fig. 7.17**)

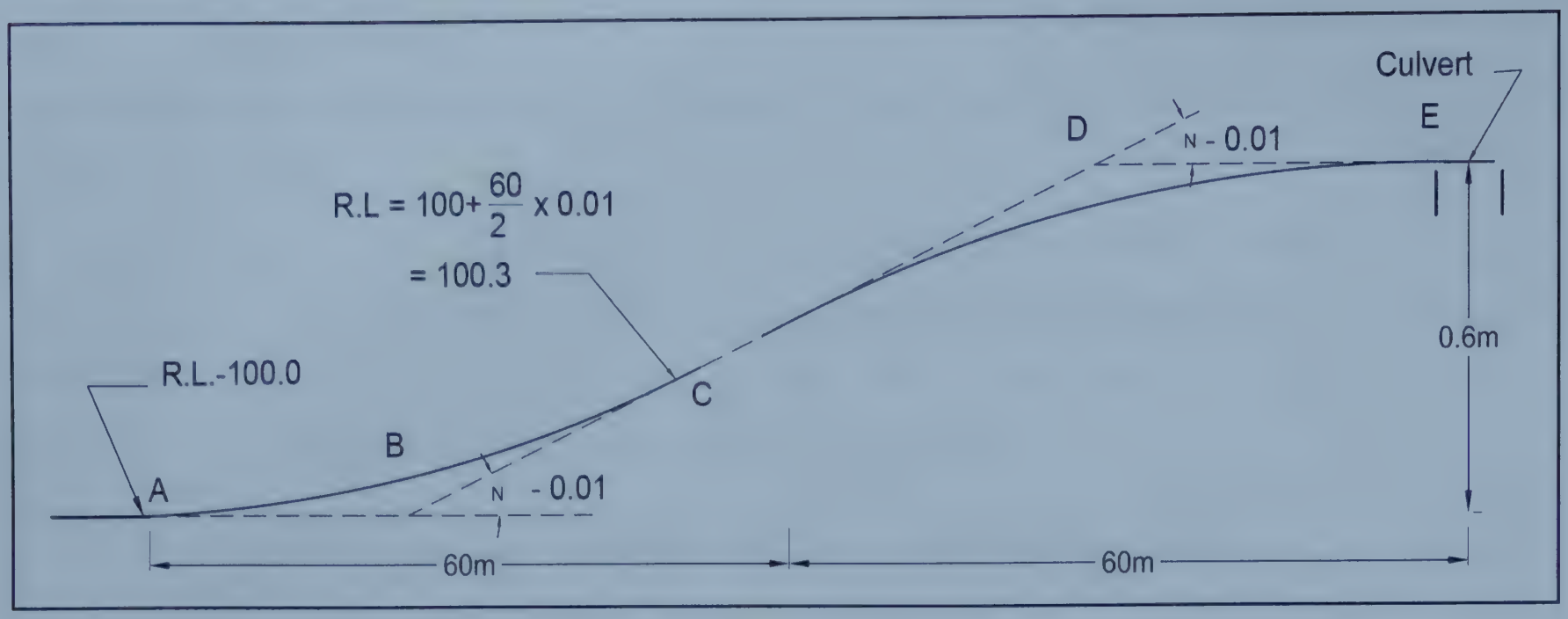


Fig. 7.17 Sketch of Problem 6

Referring to **Fig. 7.17**, let slope of the grade line be N.

$\frac{1}{2}(L_s + L_v) \times N = 0.6$

$\frac{1}{2}(60+ 60) \times N = 0.6$ (From **Fig. 7.17**, $L_s = L_v = 60$ m)

Therefore, N = 0.01

Let R.L of the road at A = 100.00 m

R.L of C = 100 + 60/2 x 0.01 = 100.30 m

R.L of B = 100 + 0.3/4 = 100.075 m

R.L of D = 100.6 - 0.3/4 = 100.525 m

Levels of in-between points as also of the approach on the other side can be calculated in a similar way.

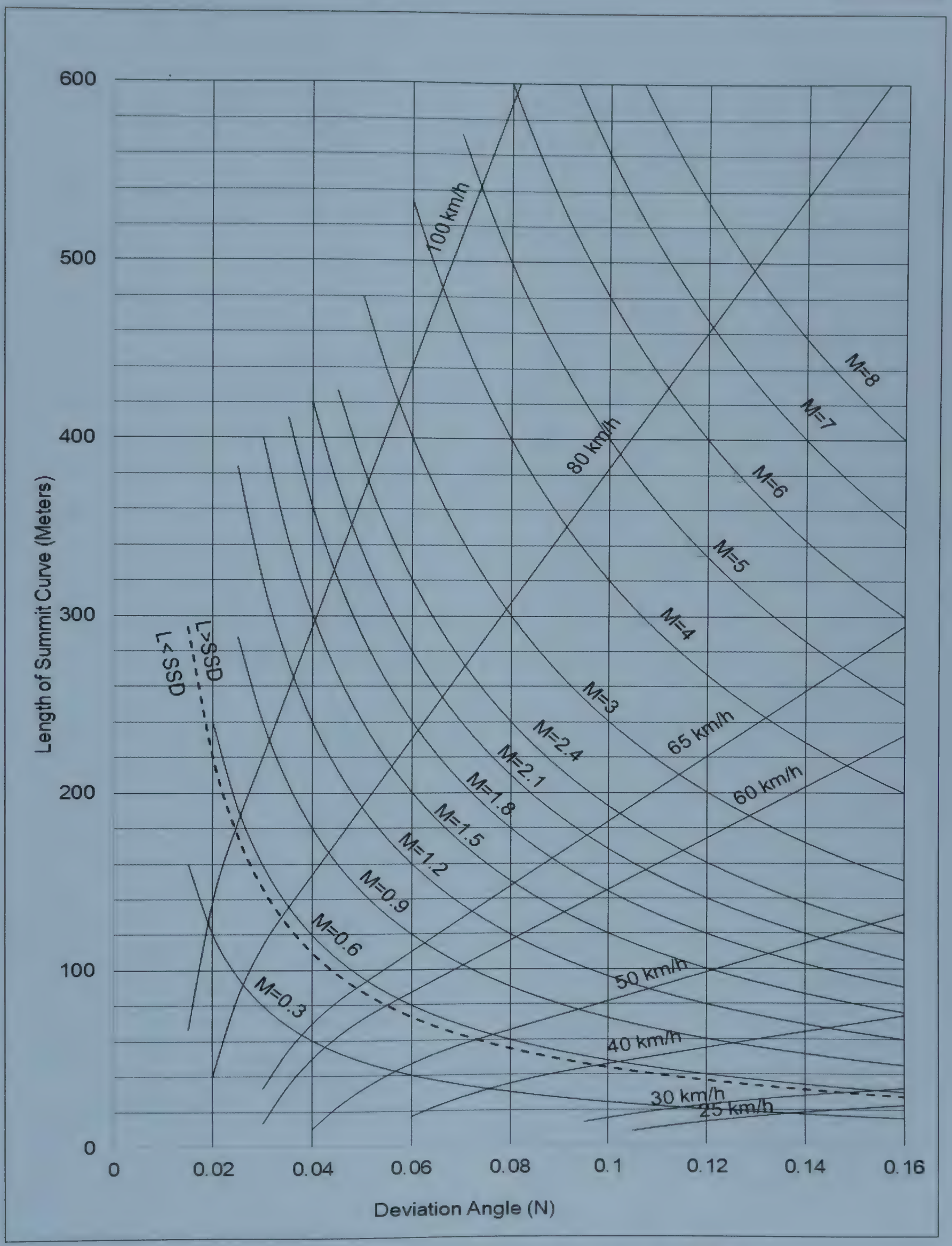


Fig. 7.18 Length of Summit Curve Considering SSD

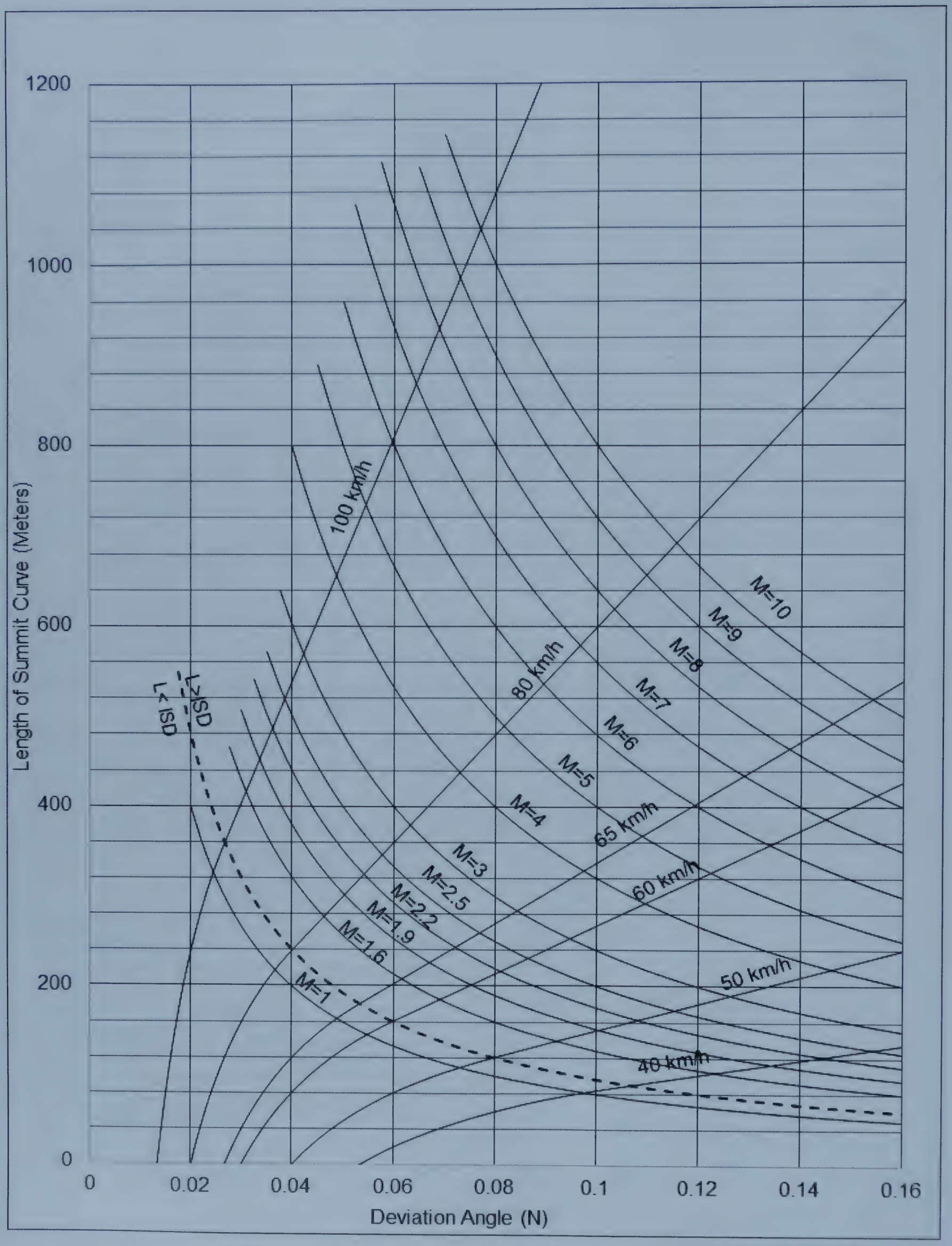


Fig. 7.19 Length of Summit Curve Considering ISD

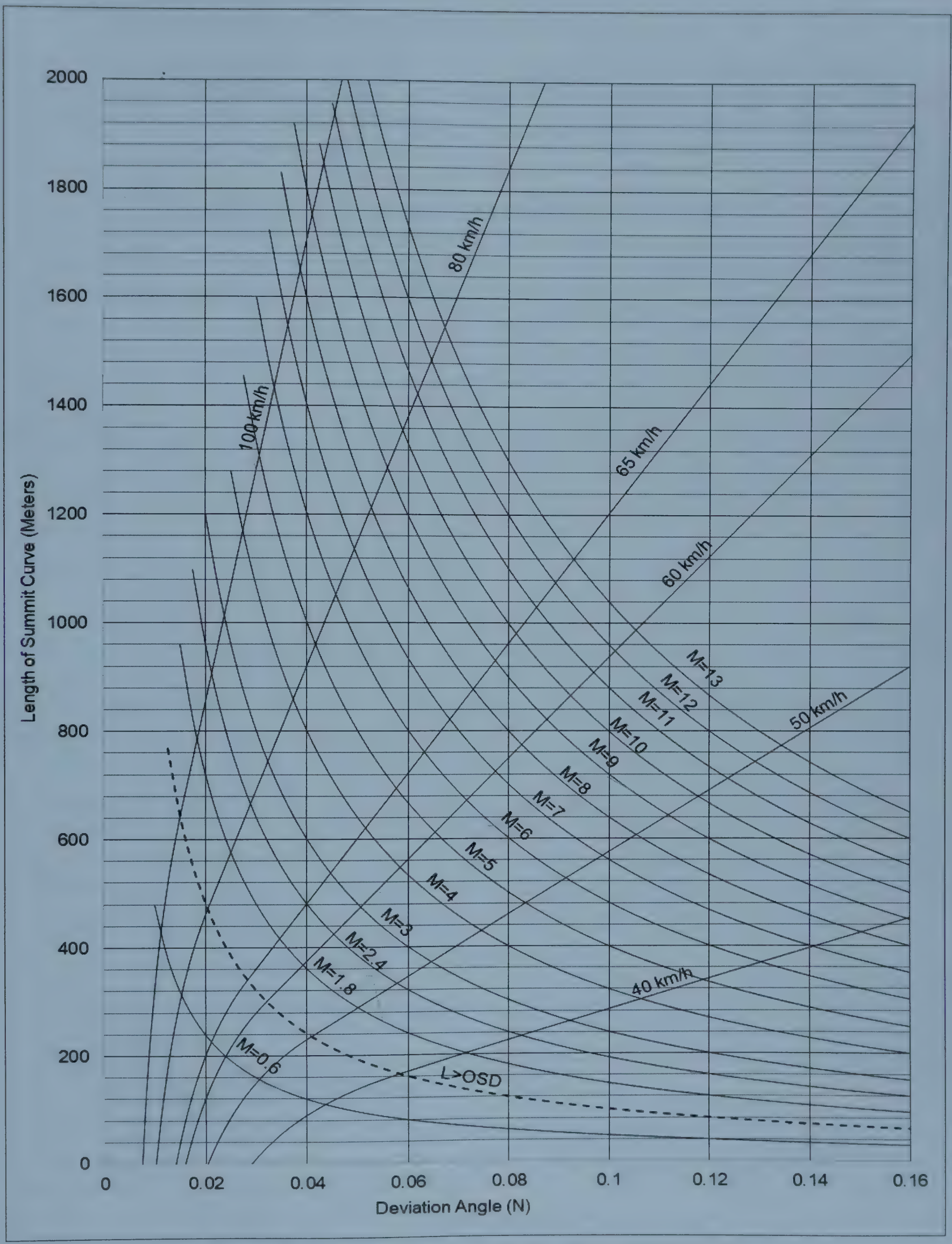


Fig. 7.20 Length of Summit Curve Considering OSD

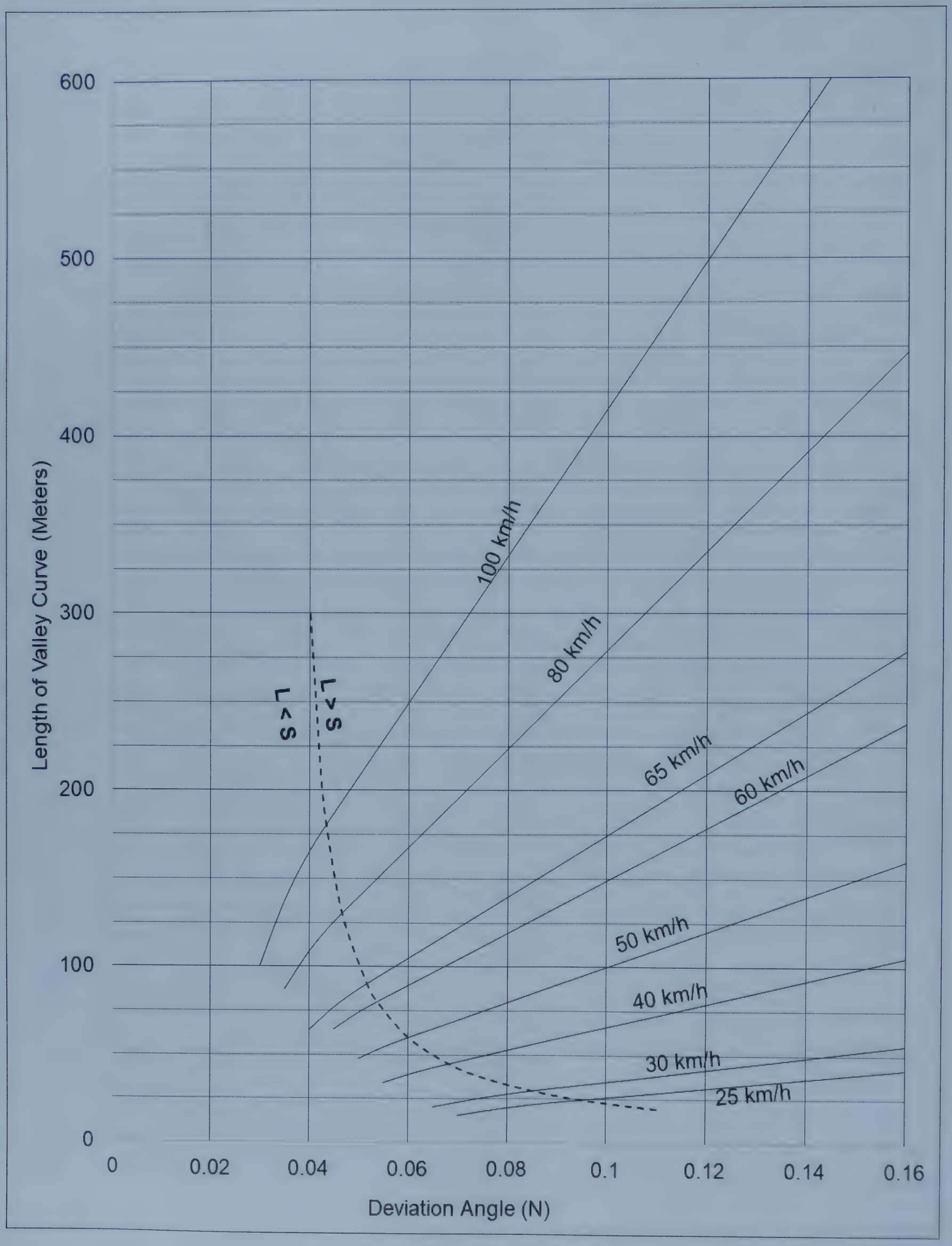


Fig. 7.21 Length of Valley Curve

7.8 Vertical Curves and Grades on Hilly Areas

In hilly areas, steep grades can contribute to large number of incidences of road crashes. This may be due to factors like insufficient sight distance, low capacity, interference to traffic on ascending grade and brake failure of the descending vehicles.

7.8.1 *Critical Length of Grades*

For safe vehicular operation on hill roads, it is appropriate to consider the length of a particular grade according to desirable vehicle operation. Critical length of grade indicates the maximum length of a particular upgrade on which a loaded truck can operate without a significant reduction in speed. By providing length of a grade lesser than the critical value, a safe and acceptable operation in the desired range of speed can be expected. For expecting desired freedom of operation on grades longer than the critical length, other design adjustments must be incorporated such as reduction in grade, and/ or providing additional lanes, etc.

Size and power of heavy vehicles (a representative truck), the speed at entrance to the curve and the minimum speed on the grade below in which interference to following vehicles is considered unreasonable, are the major factors in determining critical length. The critical length of grade shall be estimated based on the provisions as mentioned in IRC:SP:48 for designing of vertical curves for long grades, especially on hill roads.

7.8.2 *Climbing Lane on Ascending Grades*

Safety of operation and freedom of movement on two-lane highways with an upgrade are affected by slow-moving vehicles such as heavily loaded trucks. For this purpose, an additional lane (also known as climbing lane) may be added for a particular stretch of road for such slow-moving vehicles uphill so that other vehicles using the normal lane are not delayed. It must be kept in mind that a two-lane highway with a climbing lane is not considered a three-lane highway. Climbing lanes are designed for each direction independent of each other.

It is desirable to provide a climbing lane on a two-lane highway where the grade, traffic volume and heavy vehicle volume combine to degrade traffic operations from those on the approach of grade. If the traffic volume is low and delay of cars is only occasional, climbing lane although desirable, may not be economically viable. The climbing lanes shall be designed based on the provisions as mentioned in IRC:SP:48 if necessary on long grades, especially on hill roads.

7.8.3 *Emergency Escape Ramp on Descending Grades*

On long descending grades, emergency escape ramp is desirable for out-of-control vehicles, particularly trucks, to slow and stop safely. Out-of-control vehicles are generally the result of a driver losing braking ability either through overheating of the brakes due to mechanical failure or failure to downshift at the appropriate time.

Generally, three types of emergency escape ramps may be constructed: Gravity, Sand-pile and Arrester bed. The gravity escape ramps are paved or densely compacted aggregate surface, relying primarily on gravitational forces to slow and stop the runaway (such ramps are least desirable). Sand-pile ramps are composed of loose, dry sand dumped at the ramp site, are usually no more than 120 m in length. The escape ramps shall be designed based on the provisions as mentioned in IRC:52 and IRC:SP:48.

8. HORIZONTAL AND VERTICAL ALIGNMENT COORDINATION

8.1 Introduction

Individual design of a horizontal alignment and vertical alignment has been discussed in the previous chapters. There are situations, especially in mountainous and steep terrain, where in the horizontal and vertical alignment appears together or over laps each other. In such cases, even though the designs of elements are done individually, the proper fitting of the elements has to be looked at. This chapter tries to give directions on why and how the two alignments can be fitted together and with in the surrounding environment.

The overall appearance of a highway can be enhanced by judicious combination of the horizontal and vertical alignments. Plan and profile of the road should not be designed independently but in unison so as to produce an appropriate three-dimensional effect. Proper coordination or fitting together of horizontal and vertical alignment is important for achieving an esthetically pleasing design. This will ensure safety, improve utility of the highway and contribute to over all comfort of driving.

In this regard it is important to ensure optical guidance, surface guidance and spatial guidance in the coordination.

8.2 Optical, Surface and Spatial Guidance in the Coordination

a) Optical Guidance

Optical guidance means a good view of the road such that the road appears to blend with the surroundings and direction of the road becomes readily apparent. This is created by the perspective view of the road. This perspective view is to be considered only from driver's point and no other view including bird's eye view should be used for three-dimensional evaluation of the alignment. Pavement markings like edge marking, lane marking, etc. can help in improving the optical guidance. This is shown in **Fig. 8.1**.

b) Surface Guidance

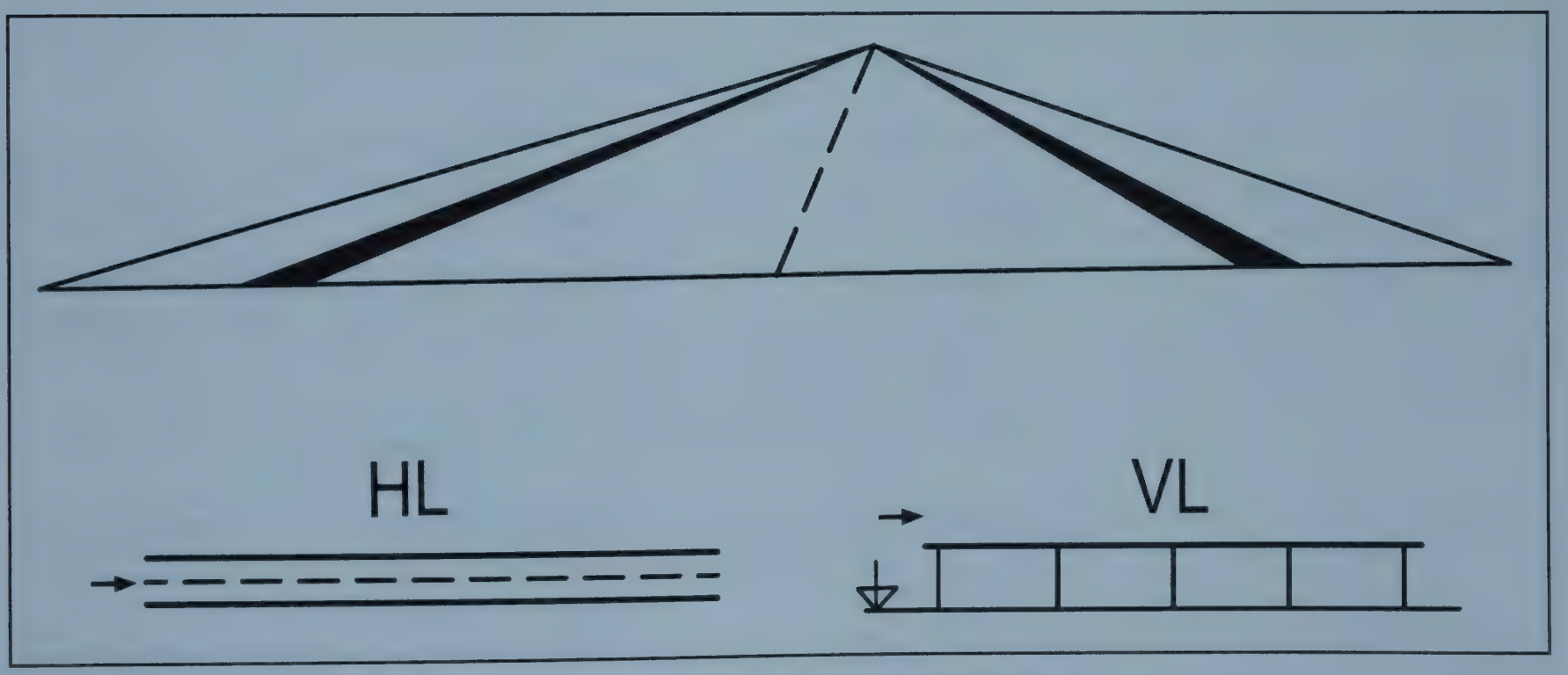
Surface guidance can be provided through the combination of cross-sectional elements of the road i.e. shoulders, carriageway width, traffic lanes, pavement markings with the design elements of horizontal and vertical alignment thus resulting in a three-dimensional design. This is shown in **Fig. 8.2**.

a) Picture of the road from the Bird's-Eye view

b) Same as a), however from the Driver's view

c) Same b), however at the beginning of the circular curve

Fig. 8.1 Bird's Eye View and Driver's View of the Alignment and Improvement on using Pavement Markings



Note: *Legend: HL= Horizontal Alignment; VL= Vertical Alignment*

Fig. 8.2 Improvement in Optical Guidance through Pavement Markings

c) Spatial Guidance

Spatial guidance is achieved through consistent sequence of three-dimensional design elements in driving space which should be balanced in terms of relationship of the design parameters among themselves. These have been shown in **Fig. 8.3**.

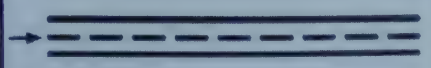
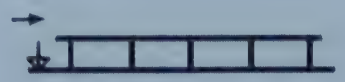
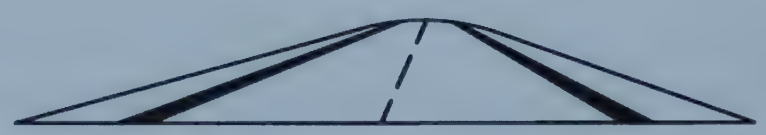
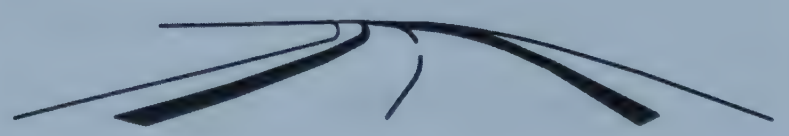
Horizontal Design Element	Vertical Design Element	Three Dimensional Design Element
 Tangent	 Tangent	Tangent with Constant Longitudinal Slope
Tangent	Curve	Straight Sag Vertical Curve
Tangent	Curve	 Straight Crest Vertical Curve
Curve	Tangent	Curve with Constant Longitudinal Slope
Curve	Curve	Curved Sag Vertical Curve
Curve	Curve	 Curved Crest Vertical Curve

Fig. 8.3 Sequential Placement of Three-Dimensional Design Elements

8.3 Three Dimensional Coordination of Design Elements

Fig. 8.4 shows some examples of bad or good three-dimensional coordination of elements. **Fig. 8.4** (a) shows a condition of shorter tangent between two horizontal curves in same direction which should be avoided. Either the tangent should be avoided and a single horizontal curve should be provided or if not possible then the length of the tangent shall be at least equal to the minimum length of the tangent. **Fig. 8.4 (b)** and **Fig. 8.4 (c)** presents a condition of short circular curve appearing between tangents.

This creates an optical break and can be avoided by connecting two tangents with along horizontal curve. **Fig. 8.4 (d)** presents a case where a short tangent is falling between two successive sag vertical curves. This is known as broken-back curve and should be avoided by provision of a long valley curve as shown in **Fig. 8.4 (e)** or by eliminating the short grade line. On similar lines, a short tangent appearing between two successive summit vertical curves should be avoided as this may give an impression of a valley curve appearing ahead. This can be rectified by provision of a long summit vertical curve. These can be seen in **Fig. 8.4 (f)** and **Fig. 8.4 (g)**.Therefore, aesthetic unpleasant effect of a short tangent can be reduced by using a sag or a valley curve with a long length and large radius. In this direction it can also be said that valley vertical curves provide best of visual qualities and optical guidance with its three dimensional design elements.

According to German design guidelines, long tangents with constant grades should be avoided. They bring in monotony in driving and make driving not enjoyable. Maximum length of the tangent (in meters) should be 20 times the design speed (in km/h) and minimum length of the tangent (in meters) should be four to six times the design speed (in km/h).

In regard to visual breaks or optical guidance it can be noted that:

(a) Optical breaks caused due to horizontal tangent or curve should be avoided. (Refer **Fig. 8.5 (a)** & **Fig. 8.5 (b)**)

(b) Optical break caused due to summit curve (as end of the road) should be avoided. (Refer **Fig. 8.5 (c)**)

(c) Partial disappearance of the road from driver's view and then its reappearance at the extended section be avoided. This is known as diving. (Refer **Fig. 8.5 (d)**)

(d) Appearance of the road on an edge of the section opposite to the direction of movement of the road should be avoided. This is known as Jumping and is shown in **Fig. 8.5 (e)**.

(e) Multiple vertical curves on along horizontal tangent or on a long horizontal curve gives a feeling of an inconsistent profile and should be avoided. This is known as fluttering and is shown in **Fig. 8.5 (f)**.

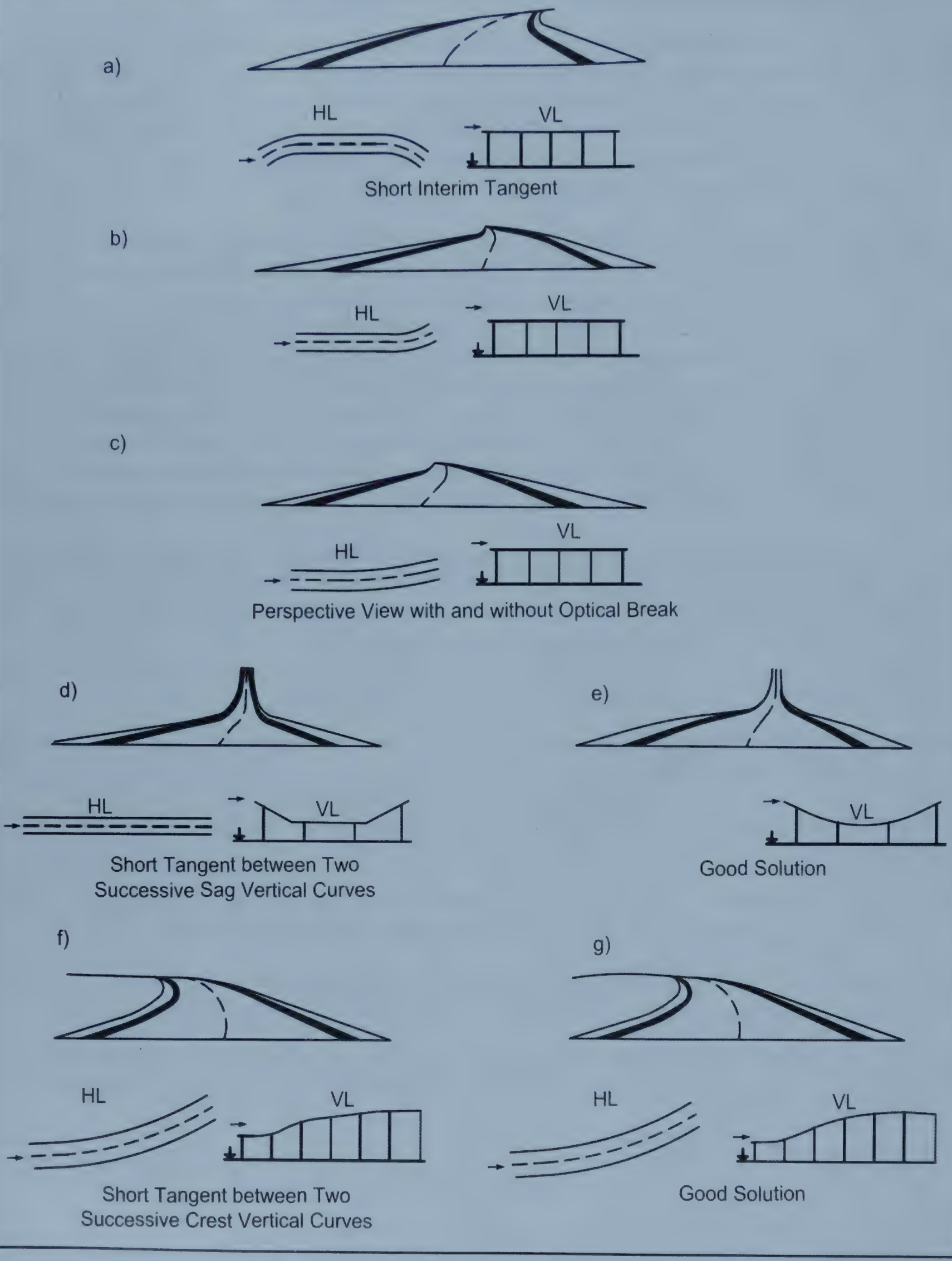


Fig. 8.4 Some Good and Bad Examples of Three-Dimensional Coordination of Elements

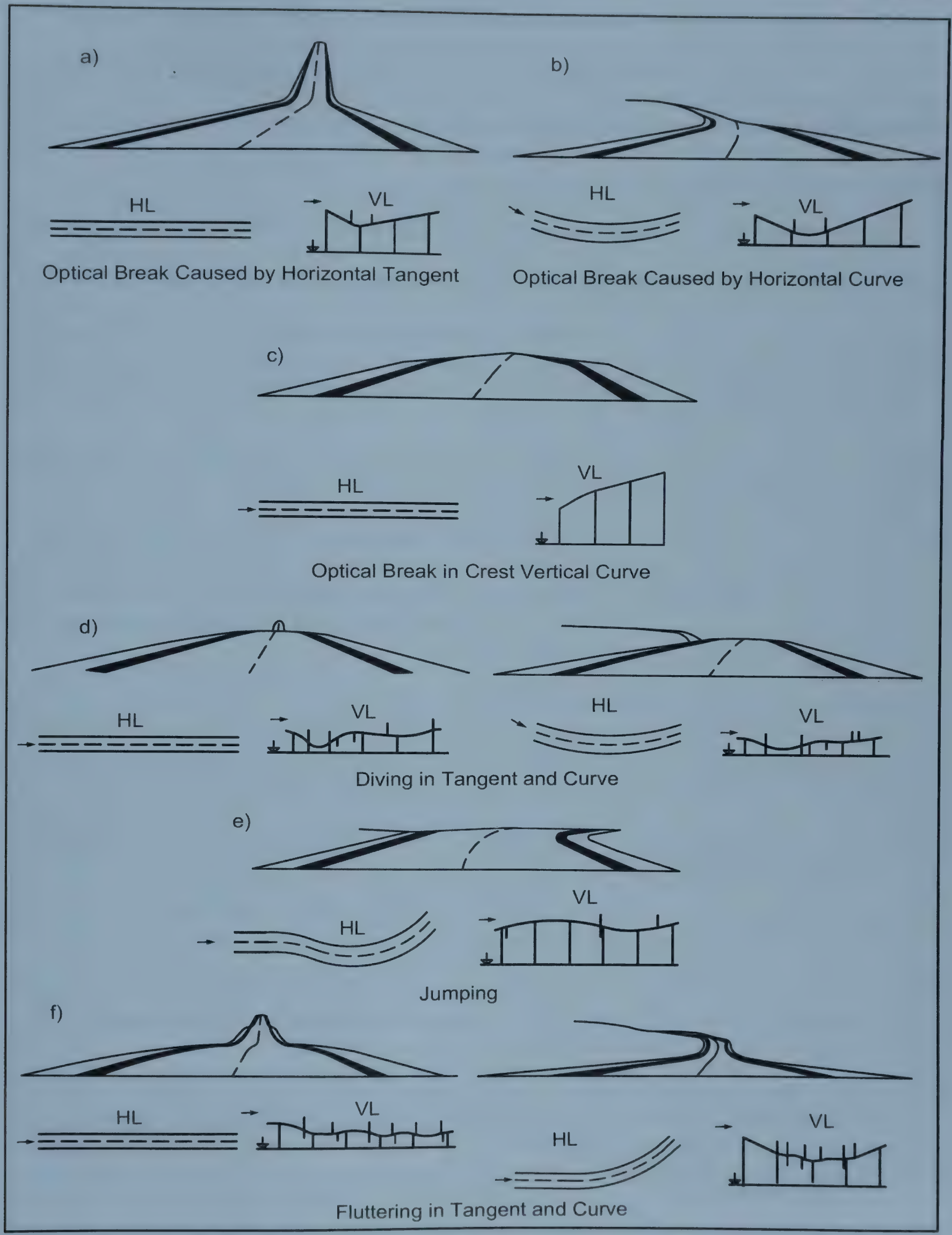


Fig. 8.5 Design Elements and Resulting Cases that should be avoided

8.4 Other Considerations for Coordination of Design Elements

The design elements of both the alignments should fuse in such a manner that they not only make the design or coordination safe but also appear to be safe to the drivers and passengers. This perception is important from psychological aspect also. Excellence in coordination of horizontal and vertical alignment increases usefulness and safety, encourages uniform speed, and improves road appearance. Other points which can be given consideration are:

(i) The degree of curvature should be in proper balance with the gradients. Straight alignment or flat horizontal curves at the expense of steep or long grades or excessive curvature in a road with flat grades do not contribute balanced designs and should be avoided.

(ii) Vertical curvature superimposed upon horizontal curvature gives a pleasing effect. As such the vertical and horizontal curves should coincide as far as possible and their lengths should be more or less equal. If this is difficult for any reason, the horizontal curve should be some what longer than the vertical curve.

(iii) The grade line for a long flat horizontal curve should not be interrupted by short dips and humps.

(iv) Ideally, the vertices of horizontal and vertical alignment or curves should coincide. A reasonable appearance will result if vertices of the horizontal and vertical curves are kept apart by not more than one-quarter phase. A shift of one-half phase will result in poor coordination and appearance. If the vertices of the two curves are slightly out of phase, the vertical curve should lie within the horizontal curve.

(v) Sharp horizontal curves should be avoided at or near the apex of pronounced summit/sag vertical curves from safety considerations. This will allow the driver to perceive change in horizontal alignment. To avoid the above problem, the horizontal curvature should lead the vertical curvature (i.e. the horizontal curve is made longer than the vertical curve).

(vi) Similarly, sharp horizontal curvature should not be placed at or near the low point of a pronounced sag/valley vertical curve.

(vii) Short horizontal curves between tangents appear as optical break. Provision of a long horizontal curve between tangents will improve aesthetics and eliminate optical break.

(viii) In hill topography, it is better to provide larger radii of crest/summit vertical curve as compared to sag/valley vertical curve. This allows longer sight distance for the crest/summit vertical curves. It in turn provides greater feeling of safety to the drivers.

(ix) In case of flat topography or where the difference in elevation along a roadway is small, the radii of sag/valley vertical curve should be larger than that of crest/summit vertical curve. This allows better spatial guidance in the form of view up to longer distance. Further, the radii of valley/summit vertical curve should be longer than the radii of horizontal curve.

(x) Short sag/valley vertical curves and crest/summit vertical curves should not appear in quick sequences.

(xi) While doing super imposition of elements the ratio between radii of horizontal curve and radii of sag/valley vertical curve should be in the range of 1/5 and 1/10. This will allow satisfactory three-dimensional appearance.

(xii) There should not be more than two changes in horizontal alignment and not more than three breaks in the vertical grade line in the view frame of the driver at any point.

(xiii) For satisfactory appearance, the horizontal and vertical curves should be longer than the minimum design standards required to satisfy safety and operational requirements.

(xiv) The appearance of the kinks can be avoided if the length of the valley vertical curve is almost same as the viewing distance from which the driver first perceived the curve. At minimum level,the length can be 0.6 times the viewing distance.

(xv) The length of the horizontal curve(in meters) should be at least 3 times the design speed (in km/h) or preferably twice that length.

(xvi) Flat spots are unsafe for road user during rains as splash of water can cause serious loss of visibility. Therefore, the carriageway of the road shall be checked for flat spot or aquaplaning for a minimum gradient of 0.5% across the carriageway so that water is effectively drained off quickly.

Fig. 8.6 illustrates some typical good and bad cases of horizontal and vertical alignment coordination in continuation of the above made discussion.

8.5 Grade Compensation

8.5.1 *General*

When horizontal curve is placed on a gradient then vehicle traversing that combination experiences two resistances, grade resistance and curvature resistance. Combined effect of these resistances will be detrimental to the performance of the vehicle. In such conditions it is advisable to compensate the gradient. The gradients should be eased by an amount known as the 'grade compensation' which is intended to offset the extra tractive effort involved at curves. This should be calculated from the following formula:

$$GC(\%) = \frac{30 + R}{R}$$

Where, R =radius of horizontal curve in meters

GC =Grade Compensation subjected to maximum allowed compensation

Maximum allowed compensation, GC = 75/R percent

Since grade compensation is not necessary for gradients flatter than 4 per cent, when applying grade compensation correction, the gradients need not be eased beyond 4 per cent.

8.5.2 *Illustrative Example on Grade Compensation*

Example: A horizontal curve of radius 200 m is laid on a grade of 5.0%. What would be the compensated gradient?

Solution:

GC = (30+200)/200=1.15 percent

Maximum allowed compensation= 75/200=0.375%

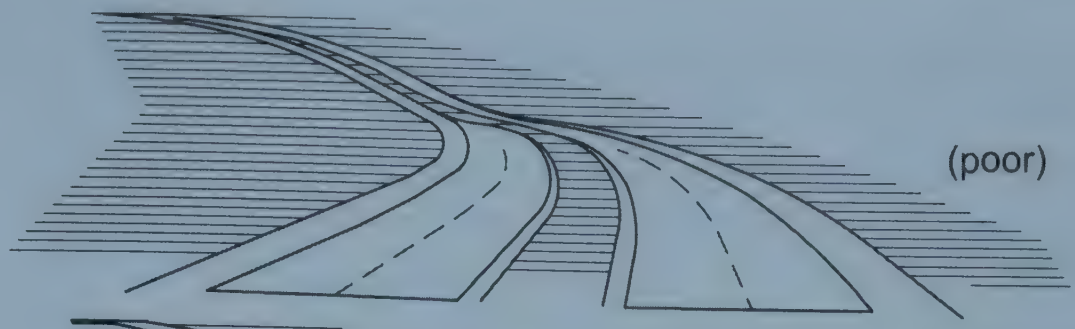
Compensated gradient=5.0–0.375=4.625%

8.6 Geometric Design Software

Most public agencies and consulting firms engaged in a large volume of highway geometric design work using highway geometric design software. Such design packages are used in conjunction with Computer-Aided Drawing (CAD) software to produce facility plans that include horizontal, vertical alignments and cross sections. The process of designing a transportation facility with a design software package usually begins with a Digital Terrain Model (DTM) obtained from topographical survey data. DTM is used to construct a contour map, slope aspect map to be used as a base map for design of road geometric elements. The design software also will usually provide for automatic stationing of alignments and fitting of standard features such as circular horizontal curves, spirals, and parabolic vertical curves. The user also defines templates of various category of road cross section. These define the geometric cross section, ditch shapes, and earthwork cross-slopes.

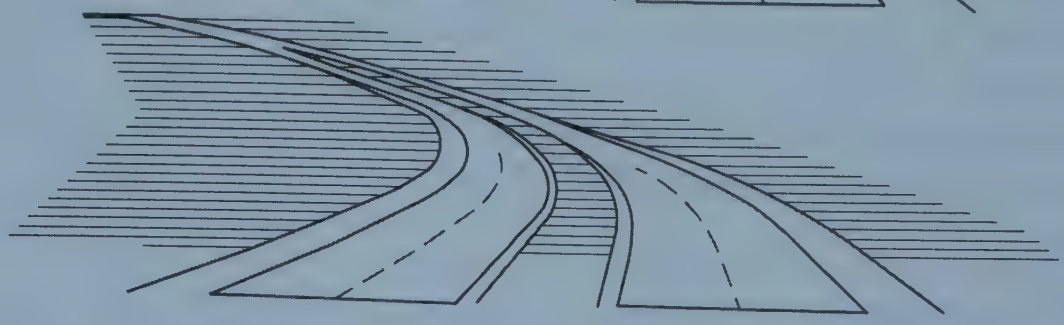
The design program has the capability of calculating transitions between successive templates in the case of multiple templates. Where there are super elevated curves, the user enters the location and other details of super elevation transitions. Once alignments and templates are defined, the program constructs a single three-dimensional model. This can be used to produce three-dimensional views. Once alignments and templates are established, the program calculates earthwork cross sections by means of a digital terrain model of base model and design model. Once earthwork cross sections are established, the design program can calculate earthwork volumes. Design software provides for rapid redesign of facilities; in particular, earthwork quantities for new alignments may be calculated very rapidly. This is a major advantage, since calculation of earthwork quantities by hand is a laborious process. Design software provides the ability to easily visualize facilities in three dimensions. Because three-dimensional representations may be constructed from any viewpoint and on any orientation, it is easy to evaluate the appearance of the facility.

a)



(poor)

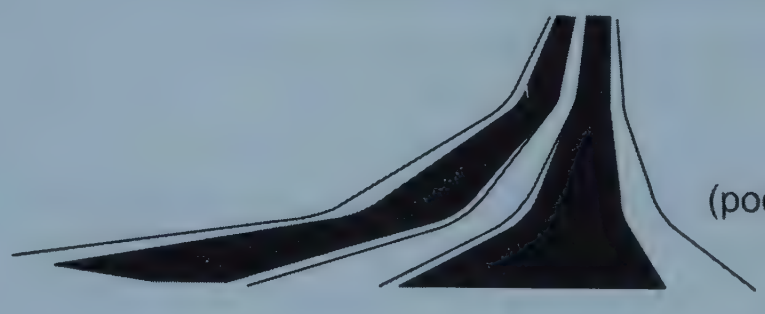
b)



(good)

(Above) Short Sag on Long Horizontal Curve
(Below) Long Sag on Long Horizontal Curve

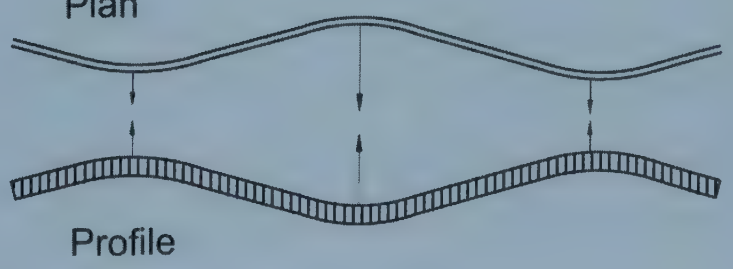
c)



(poor)

Broken - Back Vertical Curve

d)



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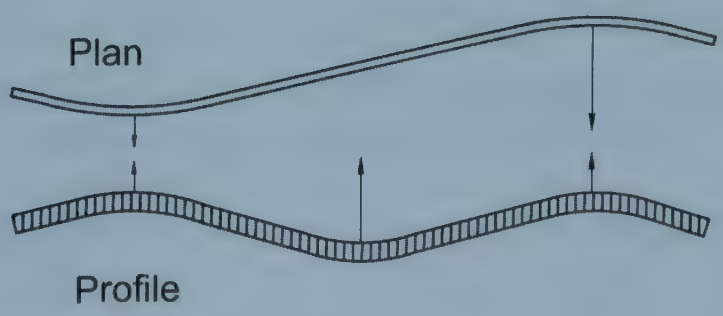
e)



(good)

f)

(poor)

Fig. 8.6 Good and Bad Examples of Coordination

9. LATERAL AND VERTICAL CLEARANCES

9.1 General

9.1.1 *Introduction*

Many times, a road has to be taken through an underpass/overpass below/above another road, railway line, and pipeline or irrigation facility like aqua duct. In order to ensure that capacity, speed and safety of highway traffic are not affected, the lateral/horizontal and vertical clearances at underpass/overpass must be adequate.

Overhead electric power and telecommunication lines crossing a road or running within the road land should be provided with adequate clearances so that safe use of the road is not affected. It is necessary to fix standards for these clearances in accordance with the maximum permissible dimensions of vehicles. Standards regarding lateral/horizontal and vertical clearances are suggested in this section. Desirable practices in this regard are indicated herein. It is recommended that these may be followed uniformly on all roads throughout the country.

9.1.2 *Definitions*

Underpass implies a short passage beneath a grade-separated structure to carry one or more streams of traffic.

Overpass implies a passage over a road through grade-separated structure to carry one or more streams of traffic.

(i) A grade separated structure which is provided for crossing of vehicles under the project highway is called as Vehicular Underpass (VUP).

(ii) A grade separated structure which is provided for crossing of light vehicles under the project highway is called as Light Vehicular Underpass (LVUP).

(iii) A grade separated structure which is provided for crossing of small vehicles under the project highway is called as Small Vehicular Underpass (SVUP).

(iv) A grade separated structure which is provided for crossing of only pedestrians under the project highway is called as Pedestrian Underpass (PUP).

(v) A grade separated structure which is provided for crossing of cattle under the project highway is called as Cattle Underpass (CUP).

(vi) A grade separated structure which is provided for crossing of vehicles over the project highway is called as Vehicular Overpass (VOP).

In general underpass may be partially/fully below ground level subjected to provision of drainage. Ideally the bed level of the crossings should be a bit higher with proper connectivity to a drain, which could serve the drainage requirements of the main carriageway, the underpass and the service road as well, however flyover needs to be above ground level. Lowering of invert level of underpass can be done with reference to HFL of nearest cross drainage structure so that connecting drain can be accordingly designed.

Lateral/Horizontal clearances

(a) For underpass/overpass, it is the distance between the extreme edge of the carriageway to the face of the nearest support whether it is a solid abutment, pier or column.

(b) For electric power and telecommunication line, the horizontal distance is measured at right angles to road alignment, between roadway or carriageway edge and a pole carrying an overhead utility line, or any pole supporting structure.

Vertical clearances

(a) For underpass/overpass, it stands for the height above the highest point of the travelled way, i.e. the carriageway and part of the shoulders meant for vehicular use, to the lowest point of the overhead structure.

(b) For electric power and telecommunication line, it is the clear vertical distance between carriageway crown and the lowest point of any overhead conductor installation which includes the conductor wire, bearer wire, guard wire, guard cradle, or screen. The lowest point should be determined after accounting for the maximum possible sag in the lowest member of the conductor installation.

9.1.3 *Overall Considerations*

(i) Conscious effort must be made to create a sense of freedom for the drivers travelling through the underpass. As far as possible, the underpass roadway should conform to the natural lines of the highway at the approaches as regards alignment, profile and cross-section. Road profile should not dip too sharply under the structure as that will produce a considerably enhanced sense of restriction when compared with a profile that proceeds smoothly through.

(ii) To promote a feeling of openness and unrestrained lateral clearance, structures with open-end spans should preferably be employed, **Fig. 9.1**. Where it becomes inescapable to have structures with solid abutments, these should be set back from the roadway edge as much as possible, **Fig. 9.2**. From considerations of cost, these treatments are meant for higher categories of roads, especially with divided carriageways.

(iii) Since lateral/horizontal clearance at an existing underpass/overpass cannot be easily increased later on, initial construction should be sufficient for the standards to which the underpass roadway would need to be upgraded within the near future. This is essential especially for important routes like national and state highways to be widened soon from single-lane to two-lane standards, also busy two-lane roads which are in the planning stage for being upgraded to a four-lane divided cross-section.

(iv) Since vertical clearance at an existing underpass/overpass cannot be easily increased later on, initial construction should be sufficient for the standards to which the underpass roadway would need to be improved within the near future to accommodate overlay or reconstruction.

(v) As a general rule, guard-rails/crash barrier must be provided on both sides of the central piers or columns, though these could be dispensed with on the abutment side when a raised foot-path forms part of the cross-section. In addition, ends of the guard-rails should be turned away from the line of approaching traffic as shown in **Fig. 9.3**, so as to deflect runaway vehicles which may otherwise hit the underpass structure.

(vi) While finalizing the invert level of underpass/overpass, the nearby HFL criteria of the water bodies (if any) shall be taken care of, i.e. the invert level shall be above the HFL for proper drainage and disposal of water from the underpass/overpass.

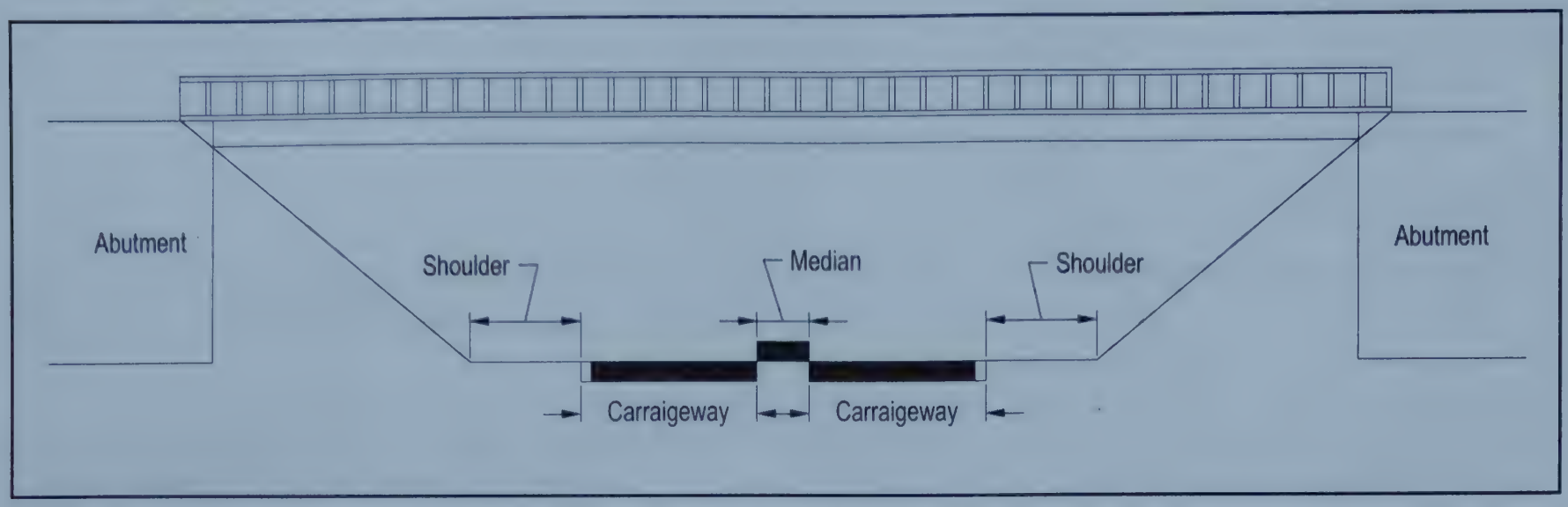


Fig. 9.1 Underpass with Open End Spans

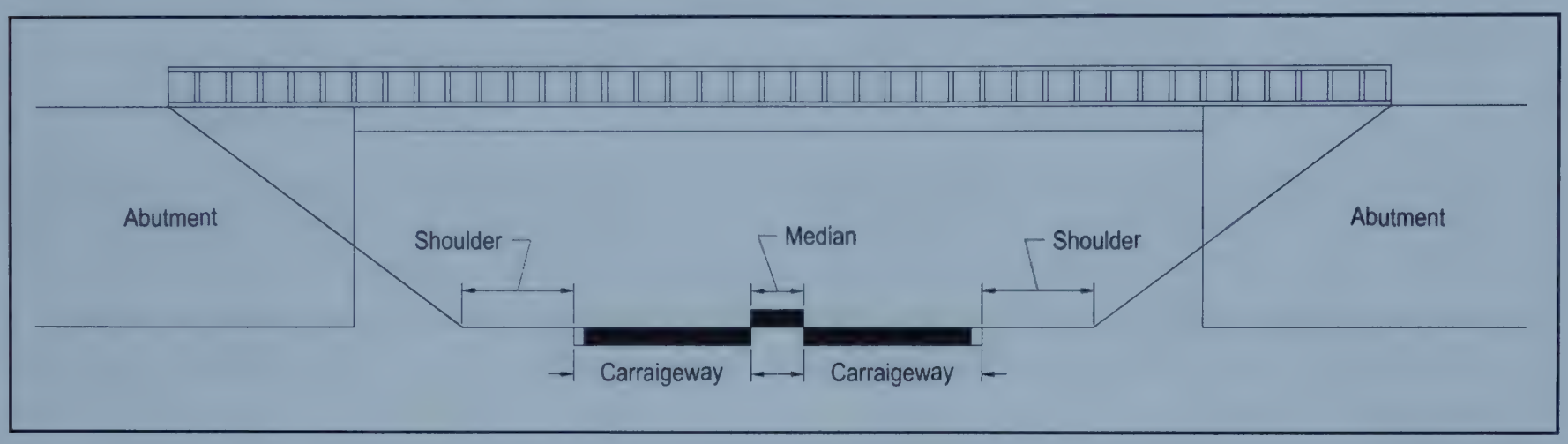


Fig. 9.2 Underpass with Solid Abutments Offset from the Shoulders

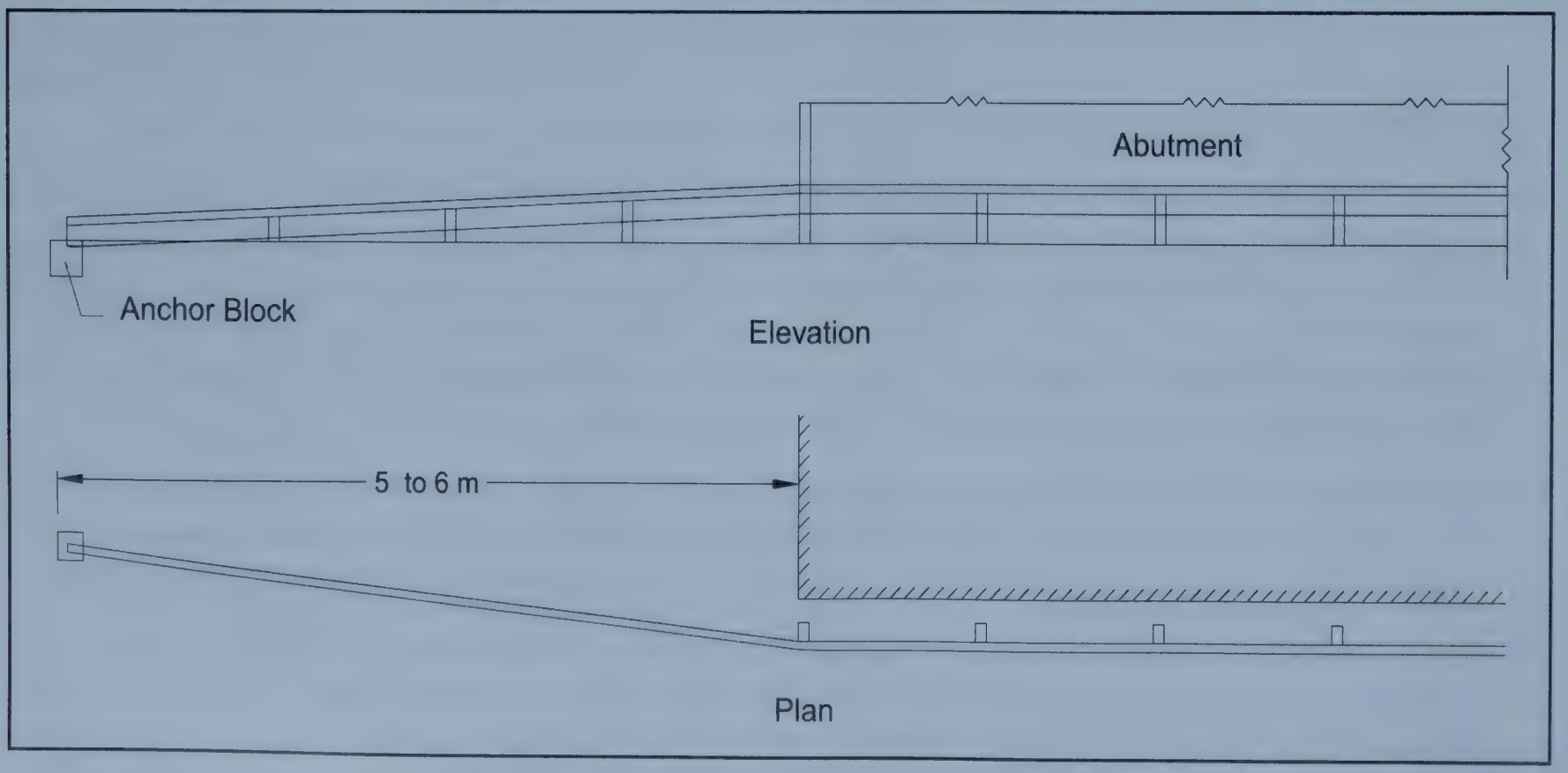


Fig. 9.3 Guard Rail End Treatment (Not to Scale)

9.2 Lateral/Horizontal Clearances

9.2.1 *Lateral/Horizontal Clearances for Underpass/Overpass*

(i) Guard rails/crash barrier shall be provided for protection of vehicles from colliding with the abutments and piers and the deck of the structures.

(ii) Full roadway width of the cross road shall be carried through the underpass/overpass; minimum lateral/horizontal span are specified in the **Table 9.1**. The minimum lateral clearance on either side must be equal to the shoulder width. This rule may be relaxed only in exceptional circumstances.

(iii) These lateral/horizontal clearances shall be applicable for all non-urban roads.

(iv) The shoulder width or lateral clearance on either side of the EW/NH/SH should be as per **section 4.4**. Similarly, the shoulder width or lateral clearance on either side of the major district and other district roads should be 2.0 meters under normal circumstances and 1.5 meters under exceptional circumstances.

(v) If a footpath is needed on a rural road, lateral clearance in the underpass portion should be the width of the footpath plus one meter. Raised footpath of minimum 1.5 m width with kerb on either side (200 mm above road surface) should be provided as shown in **Fig. 9.4 (b)**. For further details on provision on width of footpath IRC:103 maybe referred.

(vi) When a vehicular underpass is built for a divided highway, left hand side clearance shall be in accordance with **Table 9.1** i.e. shoulder width of 2.5 m. If footpaths are provided in addition, it should be as given in point - v.

(vii) Lateral clearance (shoulder width) on the right to a pier or column shall be 2 meters desirably, and 1.5 meters at the minimum as shown w_3 in **Fig. 9.4 (c)**. Where the central median is provided with kerb, the carriageway width should be increased by the side safety margin, of 0.5 meter as shown w_5 in **Fig. 9.4 (c)**. Lateral clearance in that event could be reduced to 1.5 meters (desirable value) or 1 meter (exceptional) as shown w_4 in **Fig. 9.4 (c)**. If the median is not wide enough to permit these clearances, either it should be widened gradually at the approaches or a single span structure provided across the full cross-section thereby avoiding a central pier.

Table 9.1 Minimum Lateral/Horizontal Span of Underpass/Overpass

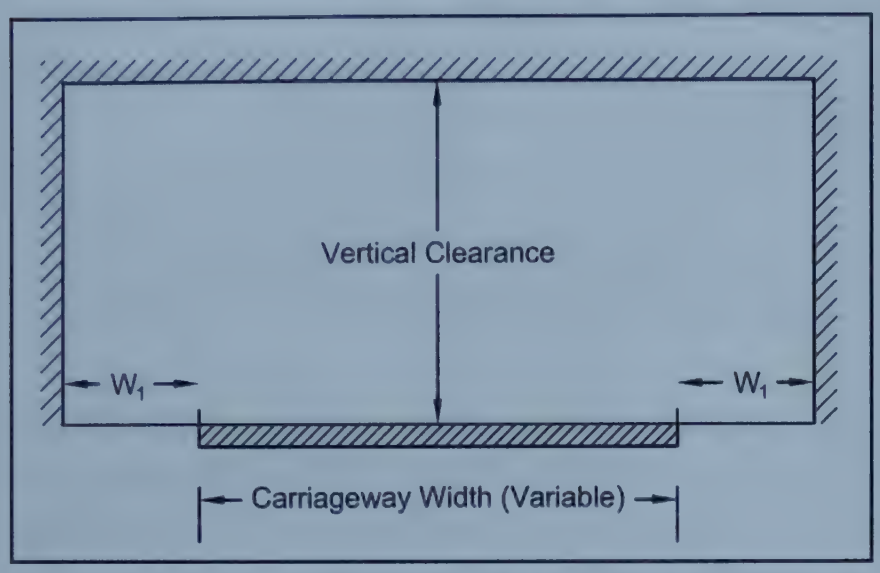
S. No	Type of Underpass / Overpass	Minimum Horizontal Clear Span (m)	Remarks
1	Vehicular Underpass	20.0	*for 2 Lane road lateral/horizontal clearance shall not be less than 12.0 m (7.0 m carriageway + 2*2.5 m shoulder width on either side)
2	Light Vehicular Underpass	12.0	*the lateral clearance shall not be less than 10.5 m including 1.5 m wide raised footpaths on either side
3	Smaller Vehicular Underpass	7.0	-
4	Pedestrian Underpass	7.0	-
5	Cattle Underpass	7.0	-
6	Vehicular Overpass	-	*full roadway width including service road and future widening

Notes:

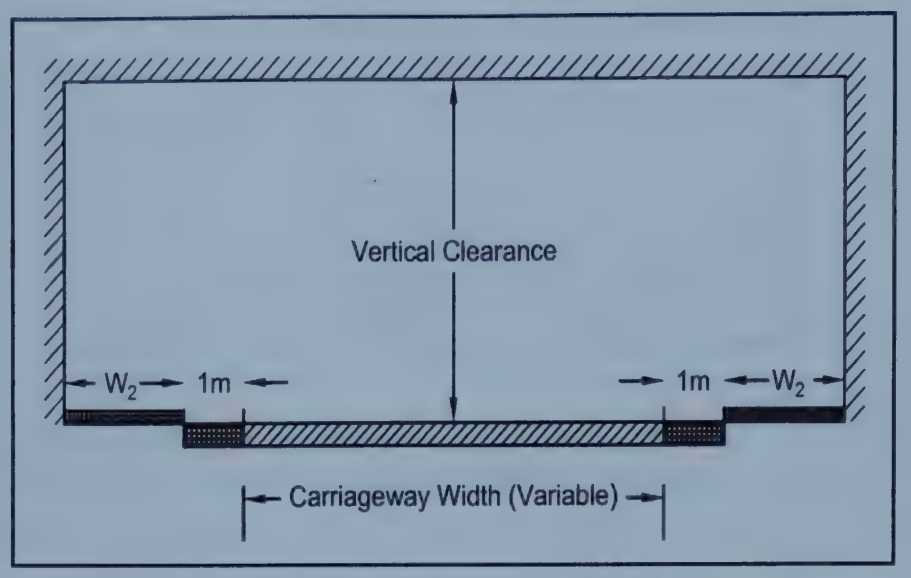
1. It is desirable that the full roadway width at the approaches should be carried through the underpass. This implies that the minimum lateral clearance (i.e., the distance between the extreme edge of the carriageway and the face of the nearest support whether a solid abutment, pier or column) should be equal to the normal roadway width.

2. The horizontal span suggested in ***Table 9.1*** *is minimum and it is suggested to adopt more liberal values than the minimum as per the project and cross-roads requirements.*

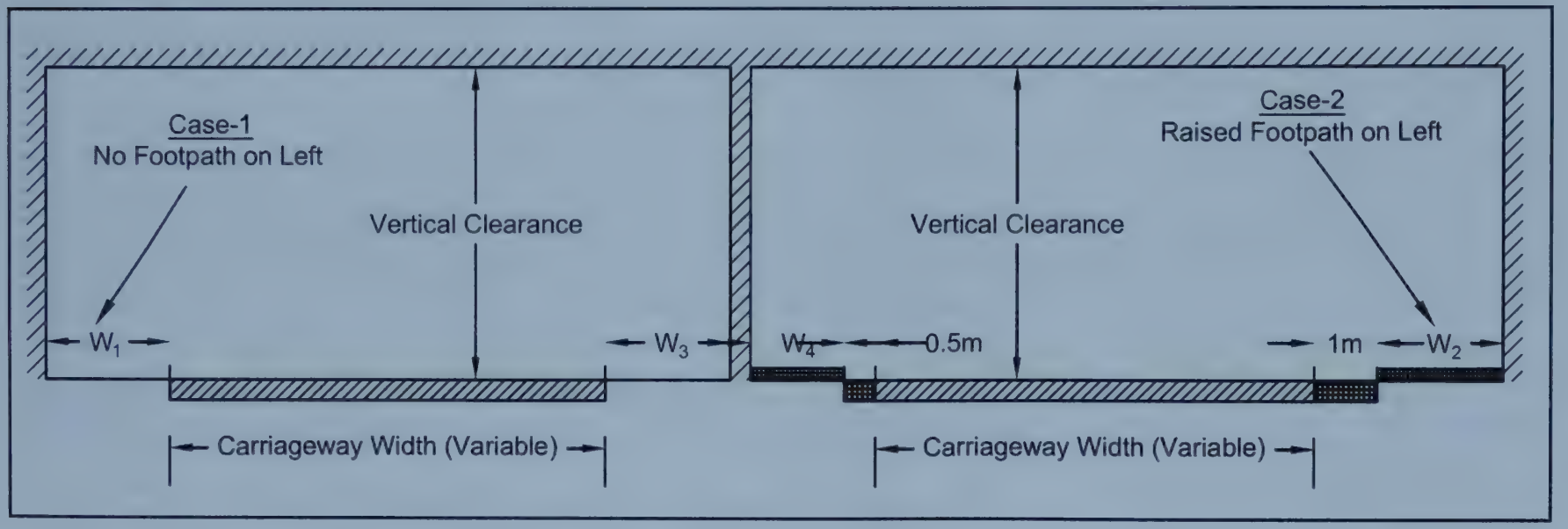
3. In case the project road passes through forest/ wildlife zones, appropriate animal passages/ underpass having sufficient length must be provided based on the type of crossing animals (such as Elephant, Camel, Tiger, etc.) as per local requirement and directions/ regulations issued by Ministry of Environment, Forest and Climate Change.



a) Single Carriageways Without Footpaths



b) Single Carriageways with Footpaths



c) Divided Carriageways

Fig. 9.4 Lateral and Vertical Clearances for Rural Highways (Not to Scale)

9.2.2 *Lateral/Horizontal Clearances for Overhead Electric Power and Telecommunication Lines*

(i) Poles carrying overhead power and telecommunication lines shall be erected at least 10.0 m away from the nearest edge of the roadway, provided also that these are at a minimum distance of 5.0 m from the nearest line of avenue trees as shown in **Fig. 9.5**. In case of roads having, at present, a narrower roadway than that prescribed in the standards in force, this horizontal clearance shall be reckoned from what will be the ultimate edge of the roadway after widening to the said standards.

(ii) The standards for lateral/horizontal clearance mentioned above shall not apply to roads situated in mountainous terrain. In such areas, poles should be erected preferably on the valley side, and as far away from the edge of the road as practicable.

(iii) The lateral/horizontal clearances in respect of poles erected for the purpose of street lighting shall be as under and as shown in **Fig. 9.6**.

(a) For Roads with Raised Kerbs - The desirable minimum lateral clearance of 600 mm should be preferred. In exceptional cases, a minimum lateral clearance of 300 mm from the edge of the raised kerb can be adopted.

(b) For Roads without Raised Kerbs- At least 1.5 m from the edge of the carriageway, subject to minimum of 5.0 m from the centre line of the carriageway.

(iv) The clearances mentioned in para (iii) shall apply to poles carrying electric power and telecommunication lines in urban situations.

(v) The clearances mentioned in paras (i) and (iii) shall be deemed to apply not only to poles but pole-supporting structures as well.

9.3 Vertical Clearances

9.3.1 *Vertical Clearances for Underpass/Overpass*

(i) In case of underpass (VUP/LVUP/SVUP/PUP&CUP), the proposed structure base shall be kept 15 cm above the ground level to ensure that these structures do not become water accumulation points.

(ii) Wherever existing slab/box culverts and bridges allow a vertical clearance of more than 2 m these can be used in dry season for pedestrian and cattle crossing by providing necessary flooring. However, these will not be a substitute for normal requirements of underpass.

(iii) These vertical clearances at underpass/overpass shall be applicable in 2/4/6/8 lane highways.

(iv) Vertical clearances at underpass/overpass shall not be less than the values given below in **Table 9.2**

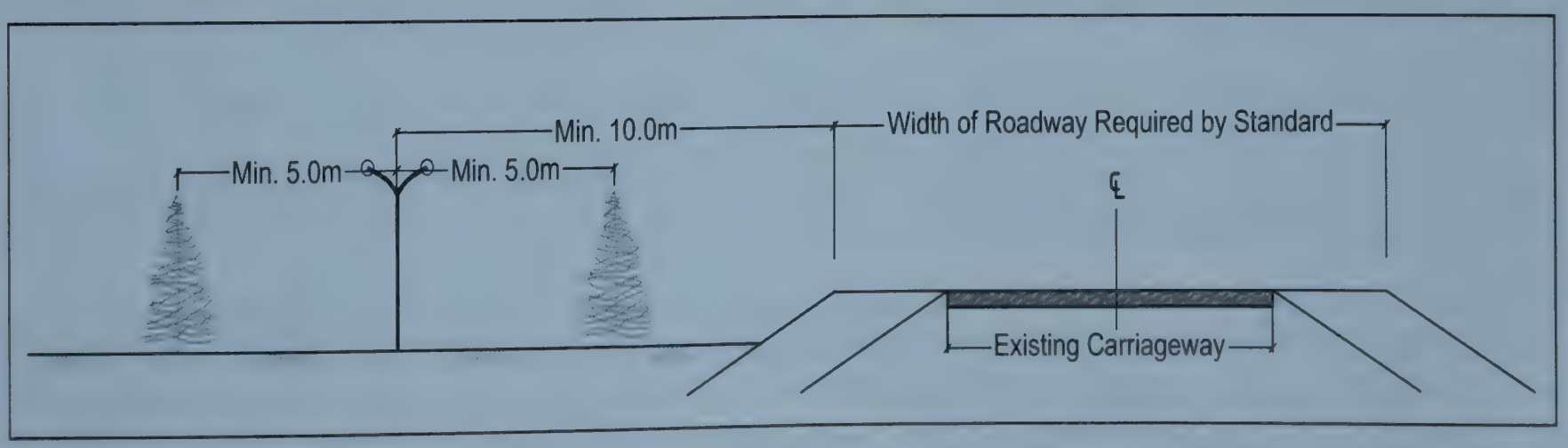


Fig. 9.5 Minimum Lateral/Horizontal Clearance for Power and Telecommunication Lines on Rural Highways, except (i) For Poles Erected for Street Lighting and (ii) For Poles Erected in Mountainous Country

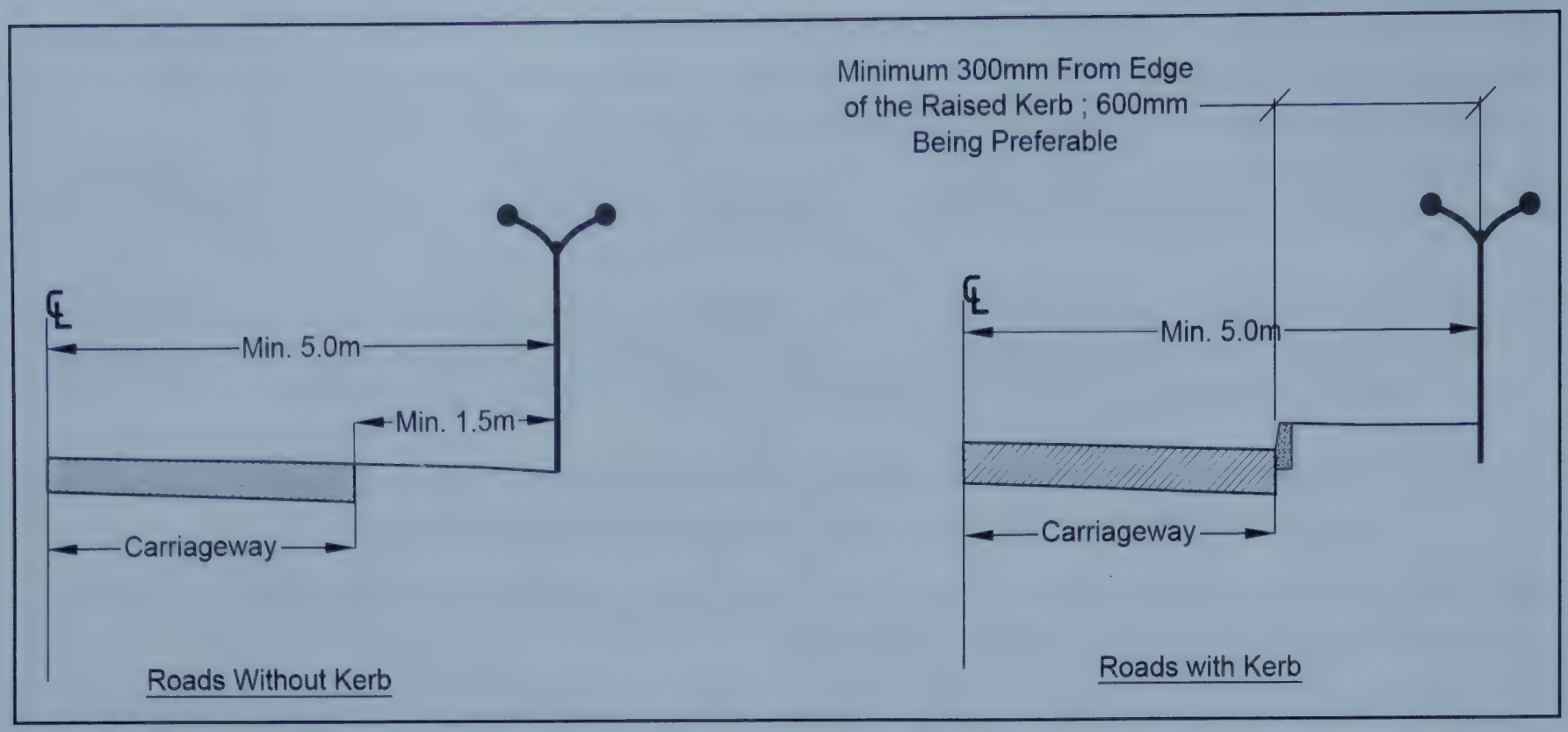


Fig. 9.6 Minimum Lateral/Horizontal Clearance for Street Lighting Poles on Rural Road and for Telecommunication, Electric Power or Street Lighting Poles on Urban Road

Table 9.2 Minimum Vertical Clearance of Underpass/Overpass

S. No	Type of Underpass / Overpass	Minimum Vertical Clearance (m)	Remarks
1	Vehicular Underpass	5.5	-
2	Light Vehicular Underpass	4.0	-
3	Smaller Vehicular Underpass	4.0	-
4	Pedestrian Underpass	3.0	-
5	Cattle Underpass	3.0	-
6	Vehicular Overpass	5.5	Clear height of 5.5 m should be available to pass the vehicle. It is defined as from maximum road level at crossing to soffit level at lowest level.

Notes: *In case the project road passes through forest/ wildlife zones, appropriate vertical clearances must be provided for animal passages based on the type of crossing animals (such as Elephant, Camel, Tiger, etc.) as per local requirement and directions/ regulations issued by Ministry of Environment, Forest and Climate Change.*

9.3.2 *Vertical Clearances for Overhead Electric Power and Telecommunication Lines*

(i) These clearances have been fixed taking into consideration the overall height of vehicles and the statutory provisions of the Central Electricity Authority (CEA).

(ii) Guard cradle or screen should be provided for electric power lines carrying voltage exceeding 110 volts while crossing the highway. The cradle should extend desirably over the full right of-way. However, guards may be omitted in the case of extra high voltage lines strung on self-supporting towers designed with adequate factor of safety.

(iii) In urban areas, in consideration of local factors such as temple cars, tazia processions, fire-fighting equipment, etc., the competent authority may prescribe higher clearances than those specified above.

Table 9.3 provides details on minimum clearance in air above ground and across road surface of highways or roads for lowest conductor of an Alternating Current (AC) overhead lines including service lines of nominal voltage system. **Table 9.4** indicates the minimum clearance in air above ground and across road surface of highways or roads for lowest conductor of High Voltage Direct Current (HVDC) overhead lines of nominal voltage system.

Table 9.3 Minimum Clearance in Air Above Ground and Across Road Surface of Highways or Roads for Lowest Conductor of an Alternating Current (AC) Overhead Lines including Service Lines of Nominal Voltage System

S. No	Nominal Voltage of System	Minimum Clearance above Ground (m)			Minimum Clearance between Conductor & Road Surface across Highway (m)
		Across Street	Along Street	Elsewhere	
1	Upto 650 V	5.80	5.50	4.60	U/G Cable
2	11 kV	6.50	5.80	4.60	U/G Cable
3	22 kV	6.50	5.80	5.20	U/G Cable
4	33 kV	6.50	5.80	5.20	11.60
5	66 kV	6.50	6.10	5.50	11.60
6	110 kV	6.50	6.10	6.10	11.60
7	132 kV	6.50	6.10	6.10	11.60
8	220 kV	7.02	7.02	7.02	12.52
9	400 kV	8.84	8.84	8.84	14.00
10	765 kV	18.00*	18.00*	18.00*	18.80
11	1200 kV	24.00*	24.00*	24.00*	30.00

Notes: **Higher clearance due to predominantly induction effects and time varying electric field (ICNIRP limit: 10 kV/m for occupational exposure) at voltage exceeding 400 kV*

Table 9.4 Minimum Clearance in Air Above Ground and Across Road Surface of Highways or Roads for Lowest Conductor of High Voltage Direct Current (HVDC) Overhead Lines of Nominal Voltage System

S.No.	DC Voltage	Ground Clearance (m)	Minimum Clearance between Conductor & Road Surface across NH/SH (m)
1	100 kV	6.50	-
2	200 kV	7.30	-
3	300 kV	8.50	-
4	400 kV	9.40	-
5	500 kV	10.60 (12.50)*	17.25
6	800 kV	13.90 (18.00)*	22.75

**Higher clearance due to predominantly time varying electric field (ICNIRP limit: 10 kV/m for occupational exposure) at voltage exceeding 400 kV*

#The vertical clearance shall be determined from the lowest point of the conductor installation which is determined after accounting for the maximum possible sag in the lowest member.

10. CONTROL OF ACCESS

10.1 Definitions

The following definitions shall apply as far as this section is concerned

Service road	:	A subsidiary road constructed between a highway/street and buildings or properties facing thereon connected only at selected points with the principal road.
Bye-pass	:	A road to enable through traffic to avoid congested areas or other obstructions to passage.
Control of Access	:	The condition where the right of owners or occupants of abutting land or other persons to access, light, air or view in connection with a highway is fully or partially controlled by public authority.
Full Control of Access	:	The authority to control access is exercised to give preference to through traffic by providing access connections with selected public roads only and by prohibiting crossings at grade or direct private driveway connections.
Partial Control of Access	:	The authority to control access is exercised to give preference to through traffic to a degree that in addition to access connections with selected public roads, there may be some private driveway connections and some crossings at grade.
Intersection	:	The general area where two or more roads/highways join or cross, within which are included the roadway and roadside facilities for traffic movements in that area.
Driveway	:	A way to secure access from a road to private property and constructed with the permission of and to the standards fixed by the highway authority and further subject to certain conditions imposed by that authority for the portion within the limits of the road land.
At-grade Intersection	:	An intersection where roads join or cross at the same level.
Highway Grade Separation	:	An intersection layout which permits crossing manoeuvres at different levels.

10.2 Need for Control of Access

Access control on a street or highway is the management or reduction of the interference with through traffic. Highway access can be managed in such a way that entrances and exits are located at points best suited to fit traffic and land-use needs and are designed to enable vehicles to enter and leave the highway with minimum interference to through traffic. This prevents vehicles from entering or leaving elsewhere such that, in spite of the type and intensity of development of the roadside areas, a high quality of service is preserved and accident potential is reduced.

If effective access control is not exercised along a highway facility, ribbon development invariably follows. Roadside businesses and small shops develop haphazardly, if allowed. Interference from the residential and commercial establishments increases, resulting in congestion. This interference is due to vehicles or pedestrians entering, leaving, and crossing the highway. Risk of accidents rises steeply due to the numerous conflicts inherent in the roads meeting the highway at numerous points. As a sequel to this, the speed drops and level of service reduces. The highway facilities become functionally obsolescent before its design life ends. The ribbon development that is going on in an unregulated way at the urban fringes of many cities should be viewed seriously if the situation is not to worsen further. Access control is one of the proven methods of combating this issue.

The control of access can be either full or partial. There is an urgent need to control the access points on national highways and state highways, if these highways have to retain their utility and not be impaired by the mushroom growth of numerous access points. The degree of access control would depend inter alia on the level of service proposed, accident frequency, legal considerations, traffic pattern, vehicle operating costs, travel time, land use, and the convenience of access to abutting property owners.

10.3 Measures to Control and Prevent Ribbon Development along Rural Roads

The preventive and control measures against ribbon development on rural roads will not only help in control of access but also in increasing mobility along the road. The following measures as per IRC:SP:15 can be adopted to prevent the ribbon developments.

(a) Acquisition of adequate highway land

(b) Planning measures for controlling land use along highways

(c) Enforcement of measures for controlling building lines, setback distances, control lines, heights of buildings, etc.

(d) Control of sub-division land abutting highway land

(e) Control of access from cross streets and private properties

(f) Control of geometric standards of access points

(g) Access controlled bypasses

(h) Service roads

(i) Removal of encroachments

(j) Accurate land maps

(k) Control of roadside advertisements

10.4 Access Management

10.4.1 *General*

Access management involves providing (or managing) access to ribbon development while simultaneously preserving the flow of traffic on the surrounding road system in terms of capacity, speed, and low crash frequency and severity.

The extent of access management depends upon the location, type, and density of ribbon development, and the nature of the highway system.

Access management is applicable to all types of roads and streets. It includes

- Setting access policies for various types of roadways
- Keying designs to these policies
- Having the access policies incorporated into legislation
- Having the legislation upheld in the courts

The road and its surrounding activities can be treated as part of a single system. Access management aims to coordinate the planning and design of each activity centre to preserve the capacity of the overall system and to allow efficient access to and from ribbon development.

Access management extends traffic engineering principles to the location, design, and operation of access roads that serve activities along streets and highways. It also includes evaluating the suitability of a site for different types of development from an access standpoint and is, in a sense, a new element of roadway design.

Access management addresses the basic questions of when, where, and how access should be provided or denied, and what legal or institutional changes are needed to enforce these decisions.

Key elements of access management include

- Defining the allowable access and access spacing for various classes of highways
- Providing a mechanism for granting variances when reasonable access cannot otherwise be provided
- Establishing means of enforcing policies and decisions

10.4.2 *Principles of Access Management*

The following principles define access management techniques:

- Functional classification of the road system by the function each road serves: Functional classification of non-urban roads is based on hierarchy of the roads based on accessibility and mobility
- Limitation of access to roads with higher functional classification: Direct access to commercial areas or a development should be low or denied along higher class roadways; while reasonable access can be provided to a lower class roadway
- Preference to through traffic movements at signalized intersections: Signalised intersection are exceptional on highways. Signalised intersection and its design should be taken into account in such a way that there is a minimal delay to through traffic
- Location of residential and commercial area entrances to minimize interference with traffic operations: It is must that entrances to ribbon development i.e. commercial

and residential areas should be located away from the major highways, intersections to minimize the accidents, traffic congestion, etc.

- Usage of channelizers, median openings, kerb medians to manage access and reducing the conflicting movements within the traffic

10.4.3 *Methods of Controlling Access*

Public agencies can manage and control access by means of statutes, land-use ordinances, geometric design policies, and driveway regulations.

a) Control by the transportation departments

Every state government and transportation department has the authority to control all aspects of highway design to protect public safety, health, and welfare.

b) Land-use ordinances

Local governments or municipalities normally administer land-use control. Local zoning ordinances and subdivision requirements can specify site design, setback distances, building and control lines, type of access, parking restrictions, and other elements that influence the type, volume, and location of generated traffic.

c) Geometric design

Access control to a road can be achieved by proper geometric design along the ribbon development in following ways

- Raised-curb medians
- The proper spacing of median openings, service roads
- Closure of median openings
- Channelization at intersections

d) Driveway regulations

State agencies or local government bodies may develop detailed access and driveway/entrance policies by guidelines, regulations, or ordinances. Policies may be formulated in such a way that a direct access to a roadway can be denied subjected to a reasonable alternative access is provided.

10.5 Guidelines for Access Control on Non-Urban Roads

10.5.1 *General*

Major corridors of inter city traffic, which are of increasing importance, need to be protected from unregulated roadside development by exercising limited access control. This is especially necessary in the case of bye-passes and highways in the urban fringes.

10.5.2 *Access to Residential Properties/Other Properties/Establishments/Driveways*

For residential properties, in urban reaches, there should be no direct access to the national highway; the access may be though the service road only.

For other properties/establishments, there shall be no direct access to the national highway. The access shall be through the service road in case of both rural and urban reaches. However, before allowing the access through service roads the possibility of providing access through nearby crossroads should be explored and only if it is feasible then the permission for access through service road at limited location maybe permitted.

10.5.3 *Access to 4-lane Highways*

Access to 4-lane highways should be partially controlled where access should be provided only at the pre-determined locations. Access to the 4-lane highways may be provided at the following locations:

- Intersection with National Highways
- Intersection with State Highways
- Intersection with Major District Roads
- Intersection with village roads and other roads, subject to a minimum distance of 3 km from the nearest intersection

10.5.4 *Access to 6-lane/8-lane Highways*

The 6-lane/8-lane highways are designed to operate at a speed dictated by the flow on the main highway and not by any other factors such as interference from local traffic, access traffic or cross traffic. Access to the 6-lane/8-lane highway shall be provided on the principle of left-in/left out arrangement and interconnected through underpasses, overpasses or grade separators. The short distance traffic, access traffic and cross traffic must be separated from the long distance through traffic. Service road shall be provided in continuous length on both sides. The continuity will be broken at (i) at locations of major bridges (ii) at locations of toll plaza in a length of 1 km, (iii) in areas where no cross traffic or access traffic is expected provided that the length of such section is not less than one km. The service roads should be connected to the main highway through properly designed acceleration/deceleration lanes or entry/exit ramps. Traffic needing access to the main highway should first come on to the service road and then join the main highway through acceleration lane/entry ramp. Similarly, all traffic exiting the highway shall first come on to the service road through deceleration lane/exit ramp from where it would distribute to the local road network. However, when there is no settlement or roadside development abutting the highway or the connecting roads are spaced more than 2 km, service roads may not be necessary and access can be given directly through acceleration/deceleration lanes.

10.5.5 *Access to Expressways*

The expressway shall be planned as a fully access controlled highway fulfilling the following essential requirements.

- Expressway should be used only by motorized vehicles
- Access must be provided only through ramps at Interchanges and specified locations: The general rule should be that access shall be provided from roads, which are one level below in the hierarchy of roads, i.e., national highways, state highways and urban arterials. In exceptional situation (e.g., where higher category roads are not available within reasonable

distance, say 10 km and there is considerable access demand), access can be provided from collector-distributor roads after upgrading them locally. All roads lower in the hierarchy of roads than expressways shall either run parallel to (i.e., without intersecting) or pass over/below the expressway (i.e., grade separated) without any connecting links (or ramps) between such roads and the expressway, except in the following situations where these are to be specifically planned:

(i) Interchange ramps and loops: Intersection with important roads (e.g., NHs/SHs) where movements in all directions are planned to and from the expressway and the intersecting roads.

(ii) Left-in and left-out ramps: Where limited movements of 'Left-in' entry to and 'left-out' exit from the expressway are planned to connect an existing road/ road network with the expressway.

- Ramp terminal only at a high category road: The terminal shall be on an NH or an SH or an urban arterial and only exceptionally on an MDR or ODR, and in no case on an access road.
- No parallel service roads: All access roads, whether existing or planned, shall be outside the functional row limit of the expressway
- Complete fencing for isolating non-expressway users
- Passages for severed communities and properties: Where property or community severance is necessitated due to ROW acquisition for the expressway suitable passage for pedestrians and vehicles across the highway shall be provided either under or over the expressway.

For more details on accessibility to expressways, grade separators and interchanges, IRC:SP:99 guidelines maybe referred.

10.5.6 *Grade Separations across Highways*

The decision to provide a grade-separated intersection should be made based on a careful consideration of a number of factors such as design consideration, safety, congestion,site topography, traffic volume, road-user benefits, etc. For a thumb-rule, following postulates can be taken as guiding principle conceptualising a grade separator. For more details, IRC:92 maybe referred.

- At all crossings of highway of the major hierarchy to be developed as fully access controlled
- At all major crossings on highways to be developed to expressway standards
- At the crossing of a major arterial road with another road of similar category carrying heavy traffic
- When an at-grade intersection fails to cater the volume of traffic resulting in congestion and frequent blockage at the intersection e.g., when total traffic of all the arms of intersection exceeds 10,000 PCU per hour
- High rate of fatal accidents at an at-grade intersection in spite of other traffic control or improvement measures
- When the topography is such that interchange is the only alternative that can be constructed economically

For detailed design guidelines of access control, IRC:62 "Guidelines for Control of Access of Highways" and IRC:SP:15 "Ribbon Development Along Highways and its Prevention" shall be referred.

10.6 Access to Fuel Stations Along Highways

10.6.1 *General Conditions for Location and Layout of Fuel Stations*

The location of the proposed fuel station should not interfere with future improvements of the highway and nearby intersections/junctions. Fuel stations shall be located at certain distance from an intersection (gap in central median to be treated as an intersection). **Table 10.1** indicates the minimum distance to be provided from intersection to fuel stations on non-urban stretches.

Table 10.1 Minimum Distance from Intersection to Fuel Stations on Non-Urban Stretches

Sl. No	Type of Terrain	Distance
1	Plain and Rolling Terrain	
	(i) Intersection with NHs/SHs/MDRs	300 m
	(ii) Intersection with Rural Roads/approach Roads to public and private properties	300 m
2	Hilly and Mountainous Terrain	
	(i) Intersection with NHs/SHs/MDRs	100 m
	(ii) Intersection with all other roads and tracks	100 m

Table 10.2 indicates minimum distance to be followed between two fuel stations along the national highway. If two or more fuel stations are said to be grouped together in close proximity, they would have a common access through a service road of 7.0 m width and connected through acceleration and deceleration lanes. The characteristics of deceleration and acceleration lanes of fuel stations are given in **Table 10.3**.

There shall not be any median gap on a divided carriageway within a distance of 300 m and 100 m on each side of the fuel station for plain/rolling terrain and hilly/mountainous terrain respectively. The minimum distance shall be measured between the start of the median gap and nearest tangent point of access/egress road of the fuel station in a direction parallel to the centreline of the nearest carriageway of the highway

Table 10.2 Minimum Distance Between Two Fuel Stations along the National Highway

Sl. No	Type of Terrain	Distance
1	Plain and Rolling Terrain	
	(i) Undivided Carriageways (for both sides of carriageways)	300 m (including deceleration/ acceleration lanes)
	(ii) Divided Carriageways (with no gap in median at this location and stretch)	1000 m (including deceleration/ acceleration lanes)
2	Hilly and Mountainous Terrain	
	(i) Undivided Carriageways (for both sides of carriageways)	300 m (clear)
	(ii) Divided Carriageways (with no gap in median at this location and stretch)	300 m (clear)

Note: *In case of divided carriageway, with no gap in medians, the distance restriction is not applicable on the opposite side of the fuel station and the minimum distance between two fuel stations on the same side shall be 1000 m.*

Table 10.3 Characteristics of Deceleration and Acceleration Lanes of Fuel Stations

Characteristics	Deceleration Lane	Acceleration Lane
Location	Take off from the edge of the paved shoulder taken upto the edge of ROW of NH beyond which boundary of fuel station will start	Take off from the edge of fuel station exit to the edge of the paved shoulder
Length	Min. 70 m	Min. 100 m with starting stretch of min 70 m having radius of 650 m and remaining 30 m tapered for merging with vehicles on main carriageway
Width	5.5 m	5.5 m
Shoulder	2.25 m (provided on the side farthest from carriageway)	2.25 m (provided on the side farthest from carriageway)

Note:

1. Wherever, available ROW is inadequate to accommodate the service roads and/or deceleration/acceleration lanes in plain and rolling terrain of non-urban stretches, the additional marginal land by the side of ROW to accommodate the deceleration/ acceleration lanes shall be acquired by the owner of the fuel station.

2. In case of widening to 4/6 lanes in near future, the matter shall be dealt on case to case basis

The fuel station should not be located within a distance of 1000 m from any barrier including that of toll plaza and railway level crossing.

No check barrier or toll plaza should be located within a 1000 m of an existing fuel station.

Fuel stations must be located at a minimum distance of 200 m and 300 m from the start of an approach road of a Road over Bridge (ROB) and the start of a grade separator/ramp/flyover respectively.

10.6.2 *Access for New Fuel Stations Along Un-Divided and Divided Carriageway Sections*

The access to the fuel stations along undivided and divided carriageway sections of national highways shall be through acceleration and deceleration lanes. The access to the fuel stations located on National highways with the service road shall be through that service road.

A separator island should be provided in front of the fuel station so that no right turning takes place. The length of this separator island would be determined on the basis of the intersecting points of the edge line of the separator island with the line drawn along the edge of chevron markings. It would have minimum width of 3 m. The width of approaches connecting deceleration and acceleration lanes, along the separator island should be 5.5 m.

There would be buffer strip from the edge of the ROW and would extend minimum 3 m inside the fuel station plot, beyond the available ROW. Its minimum length would be 12 m and 5 m in plain/rolling terrain and hilly/mountainous terrain respectively. The buffer- strip as well as the separator island would be provided with kerb of minimum 275 mm height to prevent vehicles from crossing it or using it for parking purposes.

The radius for turning curve would be minimum 13 m and that for non-turning curve be from minimum 1.5 to maximum 3 m so as to check over speeding while entering or exiting the fuel station.

Additionally, IRC:12 "Unified Guidelines for Access Permission to Fuel Stations, Private Properties, Rest Area Complexes and Such Other Facilities along National Highways" shall be used as a reference for details.

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the Code/Guidelines/Manual,etc. from the date specified therein)

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